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Outfall Sewershed Evaluation Study Plan
Project No. 1039

Sewershed Study and Plan – Final Report
Sanitary Sewer Overflow Consent Decree
Civil Action No. JFM-02-1524

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OUTFALL SEWERSHED STUDY AND PLAN
PROJECT #1039
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Executive Summary

This Sewershed Study and Plan details the evaluation of the Outfall Sewershed in the City of Baltimore. The evaluation and the contents of the report fulfill the objectives of the Consent Decree (CD), dated September 30, 2002, between the City of Baltimore (City) and the United States Environmental Protection Agency, the State of Maryland Department of the Environment and the Department of Justice. The report follows the instructions of Paragraph 9 of the CD and the guidelines of the Baltimore Sewer Evaluations Standards (BaSES) manual.

The Outfall Sewershed consists of approximately 3.6 square miles of mixed residential development and industrial area in the City. The sanitary sewer system within the Outfall Sewershed (in the City limits) is comprised of approximately 328,000 linear feet (LF) of gravity sewer lines and approximately 1,800 manholes.

The Outfall Sewershed is the most downstream sewershed in the City of Baltimore that is tributary to the Back River Wastewater Treatment Plant. As such, all of the City's sewersheds in the Back River WWTP service area are tributary to the Outfall Sewershed. These include: Jones Falls, High Level, Low Level, Herring Run, and Dundalk Sewersheds.

The main sewer system components in the Outfall Sewershed are the three large diameter trunk sewers: the 99-inch sewer, the Outfall Interceptor (arch sewer 129" x 144" and 132" x 147"), and the Outfall Relief Sewer (96" and 114"). There are no pump stations, storage tanks, or constructed SSO facilities in the Outfall Sewershed.

In accordance with the CD, the following items have been completed for the Outfall Sewershed Study and Plan:

- Evaluation of the effectiveness of the construction projects completed pursuant to Paragraph 8 of the CD using rainfall and flow monitoring data, as well as the hydraulic model developed in accordance with Paragraph 12 of the CD. There were no significant construction projects identified under Paragraph 8.

- Presentation of the results of the rainfall and flow monitoring, as well as smoke and dyed-water testing, conducted in the sewershed.

- Identification of all deficiencies discovered during the collection system inspections, which included inspection of all gravity sewers having a diameter of eight inches or greater using closed circuit television (CCTV) inspection and completed the inspection of all manholes and other appurtenances.

- Identification of all rehabilitation and other corrective actions taken, or proposed to be taken, to address the deficiencies identified during the evaluation of the sewershed.

- Description of the decision-making criteria used to select future corrective action.
- Proposal of a plan and schedule for future evaluation of the collection system within the sewershed.
- Proposal of a plan and schedule for implementing rehabilitation and other corrective actions deemed necessary either to correct deficiencies identified during the collection system evaluation or to ensure operation of the collection system without causing or contributing to an SSO.
- Determination of the range of storm events for which the collection system in its existing condition can convey peak flows without the occurrence of SSOs.
- Predictable determination of the range of storm events for which the collection system will be able to convey peak flows without the occurrence of SSOs assuming completion of the proposed rehabilitation and other corrective action projects recommended in this Sewershed Plan.
- Certification of the Geographic Information System (GIS) described in Paragraph 14 of the CD.

As required by the CD, the Sewershed Plan identifies specific improvements or other corrective actions needed to address deficiencies and aid in reducing rainfall dependent inflow and infiltration (RDII) contributing to SSOs, address deficiencies identified during the hydraulic analyses, and address other deficiencies that contribute to SSOs.

As part of the sewershed study, the City developed a condition and criticality protocol that provides the framework for a rehabilitation strategy based on criticality (consequence of failure) and condition (probability of failure) rating of 1 through 5. Assets whose failure can impact the community or environment and whose condition is the poorest received a higher rating and will receive attention sooner. Assets that receive a lower rating will receive some level of regular monitoring but no immediate action or rehabilitation. Five levels of prioritization were developed based on the combination of condition and criticality as shown in the following matrix:

<table>
<tr><th colspan="2" rowspan="2"></th><th colspan="5">Criticality</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><th rowspan="5">Condition</th><th>5</th><td colspan="5">First Priority Rehab Program</td></tr>
<tr><th>4</th><td colspan="3">Second Priority Rehab Program</td><td colspan="2"></td></tr>
<tr><th>3</th><td colspan="5">Frequent Assessment</td></tr>
<tr><th>2</th><td colspan="3" rowspan="2">Low Priority</td><td colspan="2" rowspan="2">Regular Monitoring</td></tr>
<tr><th>1</th></tr>
</table>

Figure ES-1: Condition/Criticality Matrix

Prioritization of asset rehabilitation projects and other corrective actions was developed with consideration that all proposed improvements required to eliminate SSOs must be completed before January 01, 2016, as stipulated by the CD. The proposed improvements include, rehabilitation of "First and Second Priority Rehabilitation Program" manholes and sanitary sewers, and required hydraulic improvements. The proposed improvement projects and the estimated costs to compete these repairs are summarized in the following table:

Table ES-1: Proposed Improvement Projects Summary (cost in millions of 2008 dollars)		
First and Second Priority Sewer Rehabilitation		
Rehabilitation Item	**Length/Count**	**Est. Cost**
Manhole Rehabilitation/Replacement	21 manholes	$0.1
Cured In Place Pipe Lining	85,334 LF	$6.0
Sewer Replacement	1456	$1.0
Sewer Point Repair and Cured In Place Pipe Lining	10,356 LF	$1.0
	Sub-Total Estimated Cost:	$8.1
Sewer - Hydraulic Improvements		
Project	**Amount/Size**	**Est. Cost**
Heavy Sewer Cleaning in the City of Baltimore	34,200 tons	$24.3
Heavy Sewer Cleaning in Baltimore County	29,000 tons	$20.6
Bethel Street SSO Reduction Diversion		$0.35
Total Estimated Cost for Hydraulic Improvements and Sewer Rehabilitation:		**$53.4**
Note: Costs given above do not include the downstream improvements at the Back River WWTP that are essential for the successful performance of the conveyance system in the 2-year return period event.		

The manholes and sewers that received higher condition and criticality rating scores were recommended for inclusion on the First and Second Priority corrective action plan. These repairs included the rehabilitation or replacement of 21 manhole structures, installation of over 85,300 LF of cured-in-place (CIPP) pipe liner, approximately 1,500 LF of sewer replacement, and combination of point repairs and CIPP lining for approximately 10,300 LF.

The major hydraulic improvement to be implemented in the Outfall Sewershed for the 2-year return period event is heavy sediment cleaning in the large diameter trunk sewers. In the Outfall Sewershed there are approximately 28,000 LF of large diameter trunk sewers in the City of Baltimore and 20,700 LF in Baltimore County. The sediment depth is typically 30-inches in the Outfall Interceptor. Similar sediment depths are present in the 99-inch sewer and the Outfall Relief sewer. An estimated 34,200 tons of sediment are to be removed in the City and 29,000 tons in the County.

Hydraulic model simulation results for the 2-year return period event indicate that no other facilities are needed in the Outfall Sewershed if all of the modeling assumptions are satisfied. These assumptions are:

- Downstream improvements increase the conveyance capacity to the Back River WWTP.
- After cleaning sediment, the Manning's roughness coefficient is 0.015 or less for the large diameter trunk sewers.
- The inflow boundary conditions into the Outfall Sewershed are an accurate prediction of the flows from the upstream sewersheds once hydraulic improvements are implemented in the upstream sewersheds. The upstream improvements involve conveyance restoration by sediment removal, conveyance capacity enhancements by sewer replacement, peak flow attenuation using storage tanks, and infiltration and inflow reduction by sewer and manhole rehabilitation.
- The Eastern Avenue Pump Station does not discharge more than 108 MGD (three pumps online). This means that the pump station will not operate with all pumps on line for the 2-year event.

Along with the major hydraulic improvements listed above, a short-term improvement project is recommended to reduce the risk of overflows at Bethel Street. This project is intended to attenuate the problem while waiting for the major hydraulic improvement to be implemented. To address the problem, the City is contemplating the hydraulic separation of the 24-inch branch sewer from the 99-inch sewer by diverting the flows in the 24-inch branch sewer to the Low Level Sewershed. The City has begun supplemental flow monitoring in order to characterize the peak wet weather flows in the area tributary to the 24-inch branch sewer, and will use the hydraulic model to evaluate different options to accomplish this. The additional flow monitoring and model simulations will allow the City to assess what impact, if any, this short-term measure will have on the Low Level system, and to identify additional RDII reduction projects in the tributary area as necessary.

The recommendations for hydraulic improvements in the Outfall Sewershed are subject to revision based on the Macro model results that account for the interrelationships with the upstream sewershed inflows and the downstream operations at the WWTP. System-wide Macro model results could lead to changes from the recommendations defined in this sewershed plan report for the Outfall Sewershed.

As required by Paragraph 9.C.xii of the CD, the City will also implement several continuous data collection programs in order to assess the effectiveness of the rehabilitation and other operation and maintenance enhancement efforts within the sewershed. These programs will be comprehensive, system-wide initiatives that will include a long-term flow monitoring plan, a sewer cleaning program, CCTV and manhole inspection programs and root and grease control programs.

1.0 Project Description

1.1 Project Background

On September 30, 2002, the City of Baltimore entered into a Consent Decree with the United States Environmental Protection Agency (EPA) and the State of Maryland Department of the Environment (MDE) to eliminate sanitary sewer overflows (SSOs). The objective of Paragraph 9 of the Consent Decree (CD) was to complete a series of "Collection System Evaluation and Sewershed Plans". This Sewershed Study and Plan details the evaluation of the Outfall Sewershed, one of eight Baltimore City Sewersheds.

As part of the CD, the City is required to complete a Collection System Evaluation and Sewershed Plan for each of the sewersheds within the City, including a comprehensive evaluation of all sewers 8-inches and greater. The Sewershed plans should include: sewer and appurtenance inspections to determine the existing condition of the assets and sources of wet weather inflow and infiltration (I/I), an evaluation of system capacity, and future rehabilitation and corrective actions. In August 2007, the City of Baltimore Department of Public Works (City) contracted the project team of Dewberry and Brown and Caldwell (Joint Venture) to complete a comprehensive investigation and evaluation of the wastewater collection system in the Outfall Sewershed as required under Paragraph 9 of the CD. The Joint Venture Team conducted this I/I analysis as part of the Outfall Sewershed Plan. The analysis used rain gauge and flow monitoring data provided to the City under a separate contract.

The sewershed study and plan elements are defined in the CD Paragraph 9.C as summarized below:

I. An evaluation of the effectiveness of completed and proposed construction projects using rainfall and flow monitoring data and the hydraulic model
II. Identification of all deficiencies discovered during the Collection System inspections
III. Identification of all rehabilitation and other corrective actions taken to address the deficiencies identified during evaluation of a sewershed
IV. Identification of all rehabilitation and other corrective actions proposed to be taken to address the deficiencies identified during evaluation of a sewershed
V. Description of the decision-making criteria used to select future corrective action
VI. Plan and schedule for future evaluation of the Collection System within the sewershed
VII. Plan and schedule for implementing rehabilitation and other corrective action determined necessary either to correct deficiencies or to ensure operation of the Collection System without causing or contributing to a SSO
VIII. Plan and schedule for eliminating physical connections (i.e., cross-connections) between the Collection System and the storm water collection system that allow

or have the potential to allow sanitary waste to be discharged to the storm water collection system

IX. Determination of the range of storm events for which the Collection System in its existing condition can convey peak flows without the occurrence of SSOs
X. Determination of the range of storm events for which the Collection System will be able to convey peak flows without the occurrence of SSOs assuming completion of the construction projects and completion of the proposed rehabilitation or other corrective action projects recommended by the Sewershed Study and Plan
XI. Identification of all modeled components of the Collection System that cause or contribute to flow restrictions or that have the potential to cause or contribute to overflows
XII. Presentation of results of the rainfall and flow monitoring conducted in the sewershed
XIII. Description of the quality assurance and quality control analyses performed for data collected
XIV. Description of the smoke testing and dye testing activities performed in the sewershed
XV. Quantification of the inflow and infiltration (I/I) rates and the portions of the Collection System impacted by I/I, and any identified sources of I/I to the Collection System located in the sewershed
XVI. Description of additional data collection activities that will be implemented after the completion of rehabilitation and other corrective actions
XVII. Certify that the geographic information system ("GIS") is fully functioning and capable of displaying the information described in Paragraph 14.B. of the CD

The content and structure of this Sewershed Study Report have been established to address each of the sewershed study and plan elements required under the CD. BaSES Manual (Baltimore Sewer Evaluation Standards) was created by the City of Baltimore to provide guidance to the Sewershed Consultant regarding the work required under Paragraph 9, "Collection System Evaluation and Sewershed Plan", of the CD. The manual provides broad and general guidelines, and establishes data collection/inspection criteria while setting outer boundaries that would result in consistent and uniform evaluation and preparation of sewershed plans for the entire City. The appropriate sections of the BaSES manual will be cited throughout this report to clearly identify how the development of the Outfall Sewershed Plan fulfills the requirement of the BaSES manual and the objectives of the Consent Decree.

1.2 Sewershed History/Previous Studies

<u>Sewershed History</u>

The Outfall Sewershed is a part of the wastewater collection area served by the Back River Wastewater Treatment Plant (WWTP). The western part of the Outfall Sewershed is located in the City of Baltimore and the eastern part is located in

Baltimore County where it terminates at the Back River WWTP. The western part of the Outfall Sewershed in the City of Baltimore is the focus of this report.

Early sewer systems in the City of Baltimore discharged directly to the Harbor. The Outfall Interceptor was built to convey wastewater by gravity from the City to the east, through Baltimore County, to the Back River where a treatment plant was built. The Back River Wastewater Treatment Plant (WWTP) started treating sewage in 1912 with an initial capacity of 12 million gallons per day (mgd). Wastewater from the High Level and Jones Falls Sewersheds is conveyed to the upstream end of the Outfall Interceptor. Wastewater from the Low Level Sewershed, adjacent to the harbor, is pumped into the Outfall Sewershed by the Eastern Avenue Pump station, which was constructed in 1912. The pumping station initially contained three large steam-driven pumps. Subsequent upgrades were made in 1959 and 1980. The station currently contains six pumps. Five are connected to the force main which discharges to the 99-inch pipe in Outfall Sewershed. The sixth pump is for emergency relief and discharges directly to the Harbor.

Wastewater from the Dundalk and Herring Run Sewersheds enter the Outfall Interceptor at locations further downstream prior to the County Line. The Outfall Relief sewer runs parallel to the Outfall Interceptor starting at the connection from the Herring Run Sewershed and terminating at the Back River WWTP. The Outfall Relief sewer was constructed in 1973 to accommodate these additional flows from the City and other flows from Baltimore County.

<u>Previous Studies</u>

The City of Baltimore has conducted sewershed studies since the 1970s. The 1975 “Back River Wastewater Treatment Plant Infiltration/Inflow Analysis of Wastewater Collection System,” prepared by the City of Baltimore, was an evaluation of extraneous flow entering the collection system tributary to the Back River Wastewater Treatment Plant. However, a sewer system evaluation survey (SSES) to locate extraneous flow sources was not recommended for the Outfall Sewershed at that time.

In 1992 the “City of Baltimore Facility Plan for the Back River Conveyance System” was prepared in response to requirements of the Federal Water Pollution Act of 1972, which required water pollution control funding grants administered by the USEPA. This was a system-wide study that did not focus exclusively on the Outfall Sewershed.

1.3 Purpose of Sewershed Study

The purpose of the CD is to take all measures necessary to enable Baltimore to comply with the Clean Water Act, the regulations promulgated there under, the Maryland water pollution control laws, the regulations promulgated under such laws, and Baltimore’s National Pollutant Discharge Elimination System Permits Nos. MD 21555 and MD 21601.

SSOs have been evaluated for elimination in the Outfall Sewershed collection system through development and implementation of the measures set forth in Paragraphs 8 through 15 of the CD. Illegal storm water or sewer connections are identified for elimination. Baltimore's GIS has been updated to be accurate, fully functional, and capable of displaying the information described in Paragraph 14.B.i through iv of the CD.

1.4 Description of Sewershed

The City of Baltimore (City) provides sanitary sewer services to approximately 1.6 million people within the metropolitan area. The study area known as the Outfall Sewershed consists of approximately 3.6 square miles of mixed residential development and industrial area in the City. The sanitary sewer system within the Outfall Sewershed is comprised of approximately 328,000 linear feet (LF) of gravity sewer lines and approximately 1,800 manholes.

There are no major streams or water bodies in the Outfall Sewershed. Major water users are the John Hopkins University Hospitals and industries adjacent to the railroad lines.

The Outfall Sewershed is the most downstream City of Baltimore sewershed that is tributary to the Back River WWTP. As such, all of the City's sewersheds in the Back River WWTP service area are tributary to the Outfall Sewershed. These include: Jones Falls, High Level, Low Level, Herring Run, and Dundalk Sewersheds. While the Outfall Sewershed technically extends beyond the City/County line to the Back River WWTP, by contract, the hydraulic investigation and evaluation is limited to the sewershed contained within the City of Baltimore. This contract limit was created because another concurrent study by the County on the future management of County flows is being used by the City to evaluate hydraulic influence between the County line and Back River WWTP. However, CCTV inspection was conducted on the sewer line extending from the end of the County line to the Back River WWTP. Map 1.4.1 shows the tributary Sewersheds relative to the Outfall Sewershed. Pump stations in the tributary Sewersheds convey a significant portion of the total flow delivered to the Outfall Sewershed. These pump stations are: Eastern Avenue Pump Station for the Low Level Sewershed, Jones Falls Pump Station, Dundalk Pump Station, and Quad Avenue Pump Station for a portion of the Herring Run Sewershed. There are no pump stations in the Outfall Sewershed itself.

Map 1.4.1: Location of Outfall Sewershed in the City

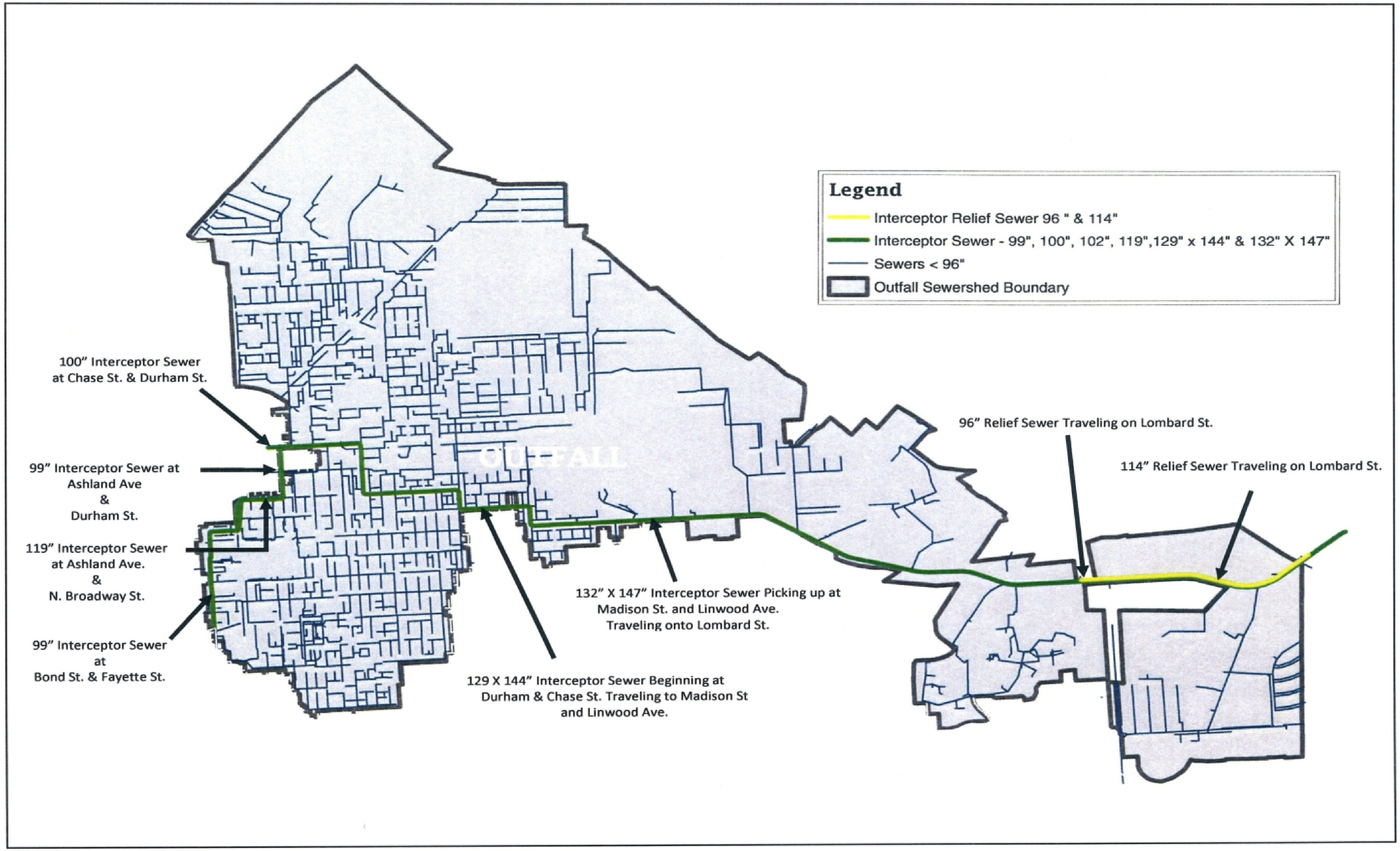
1.5 Collection System Components and Attributes

The main sewer system components in the Outfall Sewershed are the three large diameter trunk sewers: the 99-inch sewer, the Outfall Interceptor, and the Outfall Relief Sewer. There are no pump stations, storage tanks, or constructed SSO facilities in the Outfall Sewershed. The Outfall Interceptor is an arch shaped gravity sewer that runs from Chase and Durham Streets to the Back River WWTP. The Outfall Interceptor Sewer begins as a 99-inch sewer at Manhole S43C_045MH located at the intersection of Bond Street and Fayette Street. The line travels to the intersection of Broadway and Ashland Avenue where it turns into a 119-inch sewer. The 119-inch sewer extends to Rutland Avenue reducing back into the 99-inch sewer extending onto Durham Street. At the intersection of Durham Street and Chase Street, a 100-inch sewer intersects the 99-inch sewer carrying flow to an upgraded size of 144-inch sewer. This line travels to the intersection of Linwood Avenue and Madison Street where the line changes from 144-inch to 147-inch sewer extending along East Lombard Street. Map 1.5.1 illustrates the collection system components and attributes.

The Outfall Relief sewer is a circular gravity sewer that travels parallel with the Outfall Interceptor sewer along East Lombard Street beginning as a 96-inch sewer upsizing to a 144-inch sewer. The sewers are interconnected at several locations along East Lombard Street. In all, the Outfall Sewershed conveys wastewater from five other sewershed within the City.

The 99-inch sewer is a circular gravity sewer that conveys flow from the Eastern Avenue Pump Station force main to the upstream end of the Outfall Interceptor at Chase and Durham Streets.

There are several smaller branch sewers in the Outfall Sewershed that collect wastewater from the tributary areas in the Outfall Sewershed to the large diameter trunk sewers. The dominant sources of flow in the Outfall Sewershed are from the upstream sewersheds (such as the High Level and Low Level sewersheds), not the branch sewer areas in the Outfall Sewershed itself.



Map 1.5.1 – Collection System Components and Attributes

2.0 Effectiveness of Paragraph 8 Construction Projects

In accordance with Paragraph 8 and Appendix D of the CD, the City is obligated to complete a number of sewer construction projects to eliminate engineered SSO locations. There are no Paragraph 8 construction projects or engineered SSO locations associated with the Outfall Sewershed.

3.0 Flow Monitoring Program

3.1 Overall Description

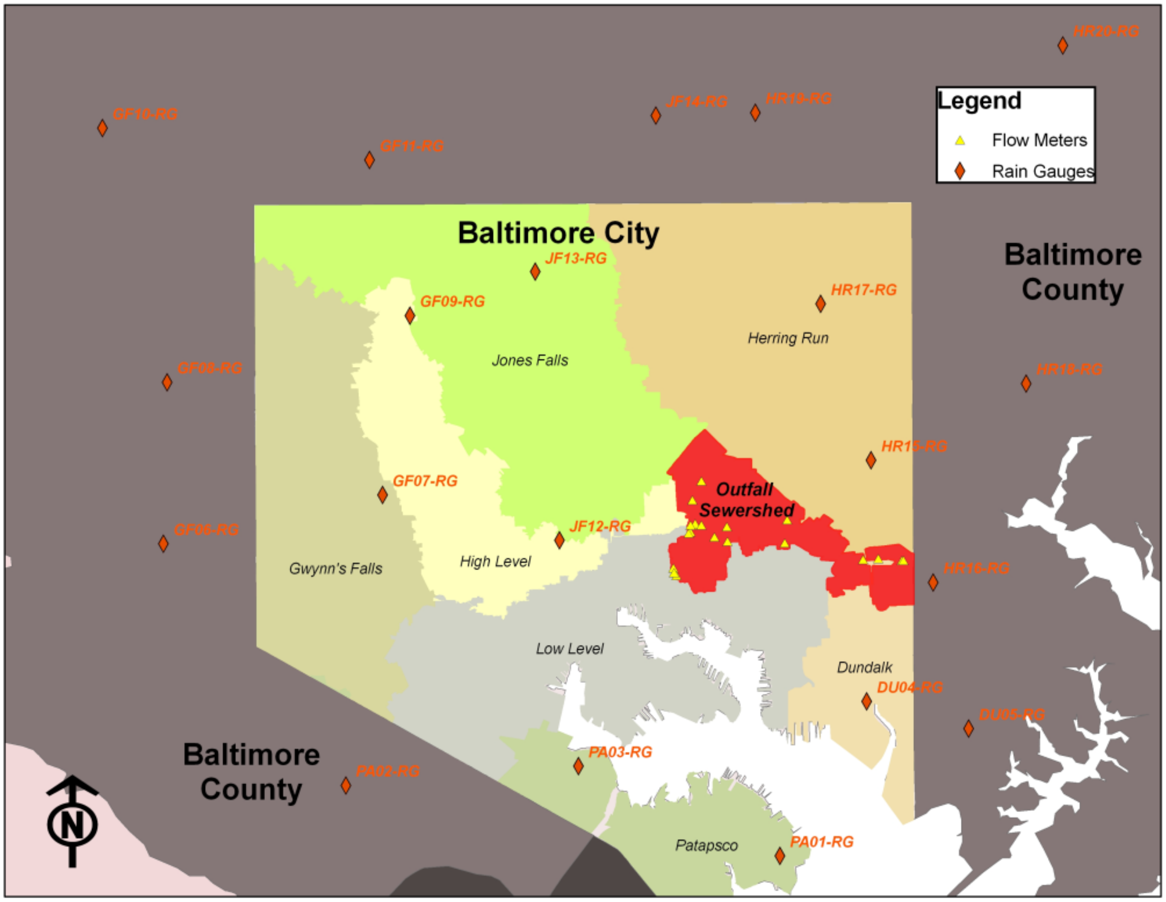
To fully understand the dynamics of the sewage collection system, the City completed a City-wide rainfall and flow monitoring program in accordance with Paragraph 9.E.-(iii) of the CD. The program consisted of flow meters within the City's collection system and rain gauges installed throughout the City of Baltimore and Baltimore County. The meters measured depth and velocity, from which flow was calculated at five minute intervals.

The objective of the monitoring program was to collect data necessary to establish the relationship between rainfall and wastewater flow in the collection system and to calibrate the hydraulic model. The flow meter locations were identified by the City in coordination with the flow metering consultant.

In the Outfall Sewershed, 22 meters were installed. In general, data was collected from May 9, 2006 to May 18, 2007. A total of five meters deemed "long term meters" continue to monitor flow. These are OUT06, TSHL01, TSOUT01A, TSOUT01B, and TSOUT02 as shown in Table 3.2.1. Using the City's Geographical Information System (GIS), the metering sites for I/I evaluation were selected at a meter density of approximately one for every 25,000 linear feet of sewer pipe.

3.2 Metering Network Within the Outfall Sewershed

Map 3.2.1 depicts the location of the meters and rain gauges within the sewershed. Figure 3.2.1 is a schematic of how the flow meters of the Outfall Sewershed are interrelated and includes some of the major wastewater conveyance features in and around the Outfall Sewershed. The flow meter names are used to identify the meter basin (or incremental meter basin) area upstream of the meter sites.



Map 3.2.1: Outfall Sewershed Flow Monitoring and Rain Gauge Network

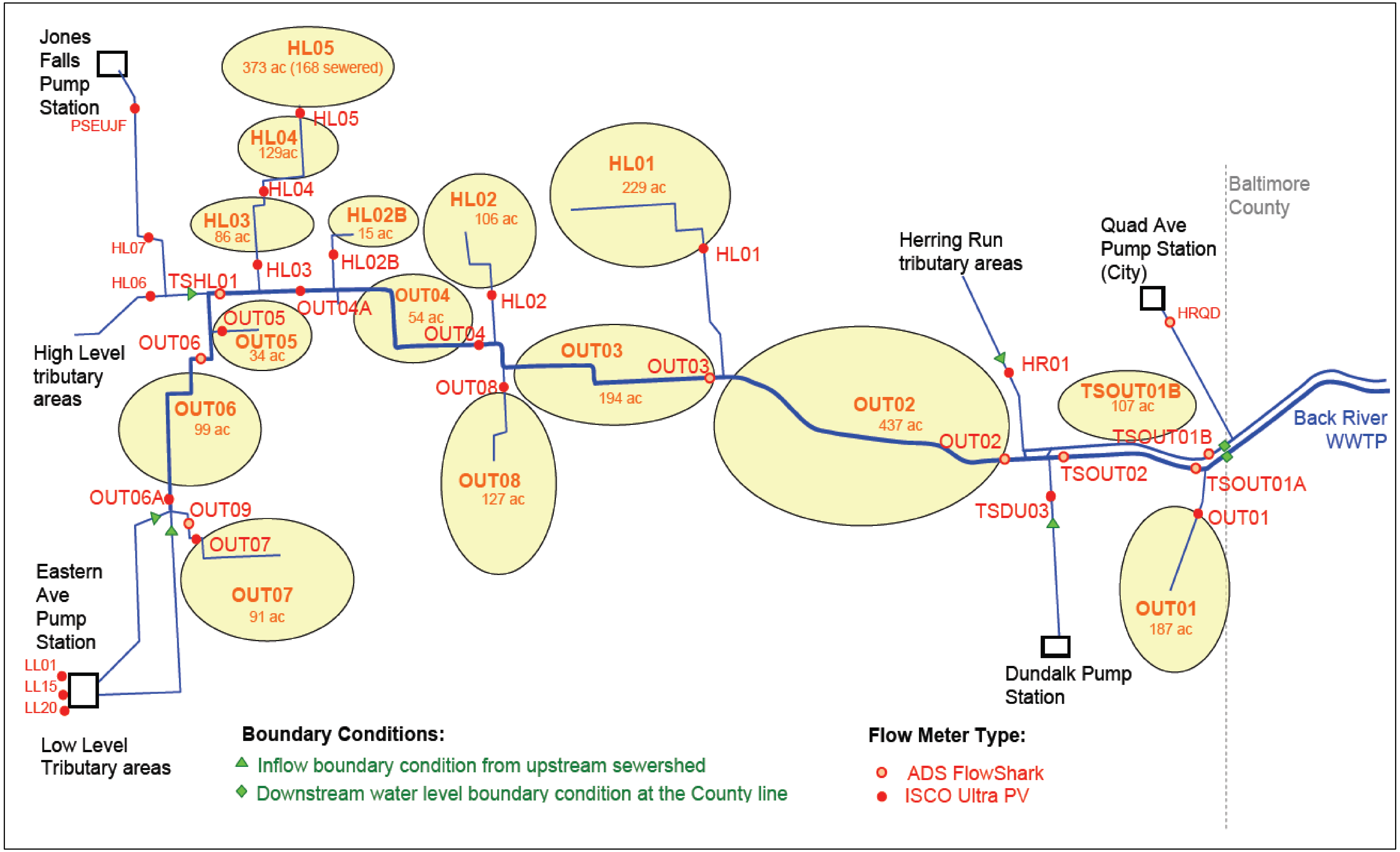


Figure 3.2.1: Outfall Sewershed Flow Monitoring Schematic

Sliicer is a data storage and analysis tool developed by ADS Environmental Services. The web based tool stores data from the flow meters, rain gauges, and radar rainfall estimates. The Sliicer tool is useful for viewing the data and determining the relationship between rainfall and wastewater flow.

Table 3.2.1 lists start and end dates for which flow meter values are available in the Sliicer database. The nominal period of flow metering extended for 374 days from May 9, 2006 to May 18, 2007. Two meters, OUT01 and OUT09, were installed several months after the other meters; therefore, the sampling durations of those meters are shorter. The permanent flow meter sites (OUT02, OUT06, TSHL01, TSOUT01A, and TSOUT01B) have data in the extended Sliicer database through March 31, 2008. Therefore, almost two years of flow monitoring data are available at those permanent sites.

Table 3.2.1 also lists how the data was used for the model development and calibration. A few of the smaller meters do not have data or the limited data is not useful for model development.

The flow-monitoring sites within the Outfall Sewershed were selected to: (1) evaluate the I/I response in the smaller branch sewers that serve the sewer service areas (SSAs) within the Sewershed. Twelve (12) meters were installed in the smaller branch sewers and Two (2) for the calibration of the hydraulic model. Ten (10) meters were installed in the major trunk sewers that convey flow from upstream Sewersheds through the Outfall Sewershed in route to the Back River WWTP.

Under project 995, flow monitoring contractors performed independent depth and velocity measurements (field confirmations or calibrations) across the full range of depths during dry and wet weather conditions throughout the project duration, assessed monitor performance relative to these measurements, and made any necessary adjustments to the equipment to maximize the accuracy of the data with respect to actual conditions. See Attachment 3.2.1 for details on flow meter field calibrations.

Table 3.2.1: Outfall Flow Meter Purpose and Installation History						
Flow Meter Site	Pipe Diameter (inches)	Flow Meter Type	Installation Purpose	Status	Start Sampling Date	End Sampling Date
HL01	24”	Isco	I/I	Temporary	5/9/2006	1/23/2007
HL02	15”	Isco	I/I	Temporary	5/9/2006	3/19/2007
HL02A	8”	Isco	I/I		NO RELIABLE DATA AVAILABLE	
HL02B	10”	Isco	I/I	Temporary	5/9/2006	4/1/2007
HL03	24”	Isco	I/I	Temporary	5/9/2006	5/18/2007
HL04	22”	Isco	I/I	Temporary	5/9/2006	5/18/2007
HL05	15”	Isco	I/I	Temporary	5/9/2006	5/18/2007
OUT01	18”	ADS Flow Shark	I/I	Temporary	2/17/2007	5/18/2007
OUT02	147” x 132”	ADS FlowShark	Calibration	Permanent	5/9/2006	3/31/2008
OUT03	147” x 132”	ADS FlowShark	Calibration	Temporary	5/9/2006	5/18/2007
OUT04	144” x 129”	Isco	Calibration	Temporary	5/9/2006	5/18/2007
OUT04A	144” x 129”	Isco	Calibration	Temporary	5/9/2006	5/18/2007
OUT05	15”	Isco	I/I	NO RELIABLE DATA AVAILABLE	NO RELIABLE DATA AVAILABLE	
OUT06	99”	ADS FlowShark	Calibration	Permanent	5/9/2006	3/31/2008
OUT06A	99”	Isco	Calibration	Temporary	5/9/2006	5/18/2007
OUT07	24”	Isco	I/I	Temporary	5/9/2006	5/18/2007
OUT08	24”	Isco	I/I	Temporary	5/9/2006	5/18/2007
OUT09	30.5"	ADS FlowShark	I/I	Temporary	8/17/2006	5/18/2007
TSHL01	144” x 129”	ADS FlowShark	Calibration	Permanent	5/9/2006	3/31/2008
TSOUT01A	147” x 132”	ADS FlowShark	Calibration	Permanent	5/9/2006	3/31/2008
TSOUT01B	147” x 132”	ADS FlowShark	Calibration	Permanent	5/9/2006	3/31/2008
TSOUT02	114”	ADS FlowShark	Calibration	Temporary	5/9/2006	2/28/2007

3.3 Rainfall Measurement

The Project 995 flow monitoring contractor was required to monitor rainfall in the vicinity of all Sewersheds using a network of rain gauges with a minimum coverage of one (1) rain gauge station per ten (10) square miles.

Map 3.2.1 shows the locations of the ground based rain gauges in the City of Baltimore and Baltimore County. None of the gauge sites are located within the Outfall Sewershed. The four closest gauges are: JF12-RG – about 1.7 miles to the West, HR15-RG – about 1.1 miles to the Northeast, HR16-RG - about 0.3 miles to the East, and DU04-RG – about 1.4 to the South.

3.4 Doppler Radar Analysis

In accordance with the requirements of the Consent Decree, Paragraph 9.E.iii.a., the City performed Doppler radar rainfall analysis in conjunction with ground-based rain gauges. Radar rainfall estimates were prepared for all rainfall events with an accumulated average total depth of greater than 0.5 inches of rain. The radar rainfall estimates are assigned to a 1 kilometer by 1 kilometer grid of "pixels"; each pixel represents a virtual rain gauge location. The flow monitoring contractor utilized the CALAMAR software platform to process rainfall events.

CALAMAR is a software and radar service package that integrates high resolution radar data with rainfall measurements collected from a ground based rain gauge network to produce accurate, dependable rainfall maps and rainfall hyetographs useful for hydrologic modeling. CALAMAR uses NEXRAD radar images from the National Weather Service along with ground based rain gauge data to transform the raw radar reflectivity images into geographically precise, local rainfall intensity hyetographs. CALAMAR uses three databases: a radar image database, a rain gauge database, and a geographical database. After collecting the rain gauge network data and the radar images, CALAMAR produced estimates of the integrated rainfall intensity for all of the one-square kilometer pixels covering the Back River and Patapsco WWTP service areas. The rainfall intensity data is stored using a five-minute interval.

A total of 29 storms during the metering period met the criteria for a global storm event as defined by the BaSES Manual Section 3.4.1. The dates of these storms are listed in Table 3.4.1. The start and end dates determine the storm period length.

Table 3.4.1 - Storms Selected for Doppler Radar Analysis

Storm Start Time	Storm End Time	Total Rainfall Depth (inches)
05/11/2006 12:00	05/11/2006 22:00	1.5
05/14/2006 23:00	05/15/2006 16:00	0.6
06/02/2006 19:00	06/03/2006 06:00	1.3
06/19/2006 14:00	06/19/2006 16:00	0.4
06/24/2006 13:00	06/24/2006 22:00	1.0
06/25/2006 04:00	06/25/2006 22:00	7.0
07/05/2006 11:00	07/06/2006 06:00	2.6
07/22/2006 14:00	07/23/2006 00:00	0.8
09/01/2006 06:00	09/02/2006 17:00	3.0
09/05/2006 02:00	09/05/2006 17:00	2.1
09/14/2006 01:00	09/14/2006 21:00	1.8
09/28/2006 17:00	09/28/2006 22:00	0.9
10/05/2006 20:00	10/06/2006 16:00	1.8
10/17/2006 07:00	10/18/2006 02:00	1.0
10/19/2006 20:00	10/20/2006 11:00	0.6
10/27/2006 15:00	10/28/2006 08:00	2.2
11/07/2006 20:00	11/08/2006 15:00	1.6
11/16/2006 08:00	11/16/2006 17:00	2.4
11/22/2006 11:00	11/23/2006 03:00	1.1
12/22/2006 12:00	12/23/2006 03:00	1.3
12/25/2006 12:00	12/26/2006 01:00	0.7
12/31/2006 16:00	01/01/2007 14:00	1.0
01/07/2007 17:00	01/08/2007 16:00	0.9
03/01/2007 18:00	03/02/2007 09:00	1.0
03/15/2007 16:00	03/16/2007 17:00	2.6
03/23/2007 13:00	03/24/2007 10:00	0.4
04/04/2007 03:00	04/04/2007 09:00	0.6
04/11/2007 21:00	04/12/2007 06:00	1.1
04/14/2007 19:00	04/16/2007 03:00	2.9

The largest event was the June 25, 2006 event with a nominal rainfall depth of 7 inches. In the vicinity of the Outfall Sewershed, this event produced approximately 4 to 4.5 inches of rain in a 24 hour duration (which is the period of rainfall that contributed to the peak flow in the sewer system). This rainfall event has a recurrence interval of approximately five years (for a 24-hour rainfall duration).

3.5 Data Collection, Processing and QA/QC Process

Under City of Baltimore Project 1015, the contractor used a host software support application program for remote wireless data collection from all flow meters, rain gauges, and ground water gauges. The raw data collected at five-minute intervals was averaged over 30-minute intervals and stored in the Sliicer database that was made available to the Joint Venture Team. All of the flow monitoring data for the modeling effort is accessed via the Sliicer software.

Sliicer is a data storage and analysis tool developed by ADS Environmental Services. The web based tool stores data from the flow meters, rain gauges, and CALAMAR radar rainfall estimates. Sliicer uses a number of data processing tools to evaluate dry weather flow, wet weather flow, and the rainfall derived infiltration and inflow (RDII) characteristics that can be derived from the flow meter data. The Sliicer tool was used for viewing the data, reviewing the correspondence between rainfall and wastewater flow, and developing preliminary model parameters that was be utilized in the InfoWorks™ software.

The Project 995 flow metering contractor was required to collect useable flow data a minimum of 90% of the time throughout the nominal monitoring period. In the event that depth measurements are available but velocity measurements are missing, the uptime requirement may be satisfied by inferring velocity from a reliable depth measurement. The flow metering contractor was required to identify all inferred velocity data or other data derived from inferred data in all reports and deliverables.

The Joint Venture Team screened the values in the Sliicer database to determine the percent of non-zero data values (for flow, depth, and velocity) in the actual sampling duration. The percentage of non-zero values is based on the actual sampling duration of each site, not on the nominal monitoring program duration. The uptime percentage information was prepared by the City's technical program manager. Flow meter and rainfall data was used to calibrate the hydraulic model. The interpretation of the flow meter data is discussed in Section 5 below in relation to the model calibration process.

3.6 Dry Weather Analysis

The objective of the dry weather flow development is to characterize the dry weather flow pattern so that during wet weather conditions it is possible to distinguish between flow due to I/I and the BSF. Following the criteria established in BaSES Manual Sections 3.2.3 and 3.5, dry days were defined.

The dry day groups were separated into weekdays and weekends since diurnal patterns of these groups are often quite different. The weekdays include Mondays through Fridays. The weekends include Saturdays and Sundays. Season groups are discussed in the BaSES Manual Section 3.5.4. The seasons used for the study were defined by the use of Eastern Daylight Time (EDT) and Eastern Standard Time (EST).

GWI was normalized by the inch-diameter-miles (IDM) to take into account not only the length, but also the diameter of the pipes in the basin. The pipe surface exposed to infiltration is proportional to the length of pipe and the diameter. IDM is a metric that is proportional to the surface area of the pipe potentially exposed to groundwater.

3.6.1 Base Infiltration Rates and Severity

The dry weather flow data was evaluated to estimate the ADF, BSF, and GWI in accordance with BaSES Manual Sections 3.5 and 7.4.5. During dry weather conditions, the flows vary with diurnal and weekly patterns. The results presented in this section are the averaged values for typical weekday conditions.

Table 3.6.1 is a summary of the dry weather flow results along with selected meter basin characteristics (e.g., gross area, net area, IDM, pipe length). The ADF values are given for both the gross and net flow rates. For terminal meters on branch sewers, the gross and net ADF values are identical. Meter basins HL03 and HL04 have net ADF values derived from subtracting the flow from the upstream meter along the branch sewer. The meter basins along the large Outfall Interceptor have gross ADF values based on meter data, but the net ADF values are assumed values that are used in modeling. For these meter basins, the ratio of net ADF to gross ADF is very small (less than 1%). Because the net ADF is a very small percentage of the gross ADF, it is not possible to estimate accurately the net ADF from the meter data.

The net BSF values are based on the SSA flows provided by the City based on water consumption records. The net GWI values are estimated from the flow meter data by subtracting the net BSF from net ADF. When flow meter data could not be used to estimate net GWI values, a nominal value equal to the net BSF was used; this is the case for all of the meters on the large trunk sewers and two of the branch sewers that did not have useful meter data. The net BSF is normalized by dividing by the length of pipe in the meter basins. The net BSF rates range from 1.9 to 13.9 gpd/LF; with the majority between 5 and 10 gpd/LF. The highest rate (13.9 gpd/LF) is in the OUT06 meter basin, which is likely influenced by the John Hopkins University Hospital round-the-clock operation.

The severity of the GWI is a normalized value in which the net GWI was divided by the IDM value for the meter basin. Meter basins OUT01 and HL02 have the highest values for GWI severity. Meter basins HL01, HL03 and OUT08 have moderate values, while HL04, HL05, and OUT07 have relatively low values for GWI severity. These results are presented graphically in Map 3.6.1 where GWI severity values are shown by meter basin. The GWI values are also listed in Table 5-1 as a percentage of ADF. There is no monitoring data for the ground water table in the Outfall Sewershed. The flow meter data does not suggest that GWI has a significant seasonal variation that may be related to fluctuations in the ground water table. Therefore, it is assumed that the GWI is relatively constant throughout the year.

Table 3.6.1: Dry-Weather Analysis Table

Basin	A_{gross} (1) (acres)	A_{net} (2) (acres)	A_{net}/A_{gross} (%)	IDM (in-dia-mile)	Pipe Length (LF)	ADF_{gross} (3) (MGD)	ADF_{net} (4) (MGD)	ADF_{net}/ADF_{gross} (%)	BSF_{net} (5) (MGD)	GWI_{net} (6) (MGD)	GWI Severity (gpd/idm)	GWI_{net} / ADF_{net} (%)	BSF_{net} Rate (gpd/LF)
HL01	228.7	228.7	100%	46.2	22227	0.30	0.30	100%	0.141	0.159	3440	53%	6.4
HL02	106.3	106.3	100%	44.6	28040	0.54	0.54	100%	0.205	0.335	7510	62%	7.3
HL02B	15.2	15.2	100%	6.2	4057	0.06	0.06	100%	0.030	0.030	Assumed	50%	7.3
HL03	383.3	86.4	23%	41.8	23061	0.79	0.28	35%	0.147	0.133	3190	48%	6.4
HL04	296.9	128.8	43%	51.9	29619	0.51	0.22	43%	0.185	0.035	680	16%	6.2
HL05	168.2	168.2	100%	45.0	26614	0.29	0.29	100%	0.256	0.034	760	12%	9.6
OUT01	134.0	134.0	100%	30.0	17236	0.40	0.40	100%	0.095	0.305	10180	76%	5.5
OUT02	NA	437.4	NA	225.7	27209	78.00	0.29	0.4%	0.144	0.144	Assumed	50%	5.3
OUT03	NA	194.3	NA	228.2	34920	77.00	0.68	0.9%	0.338	0.338	Assumed	50%	9.7
OUT04	NA	54.4	NA	77.8	10189	75.00	0.21	0.3%	0.106	0.106	Assumed	50%	10.4
OUT05	33.8	33.8	100%	14.2	8766	0.17	0.17	100%	0.084	0.084	Assumed	50%	9.6
OUT06	NA	99.1	NA	97.5	17690	26.00	0.49	1.9%	0.246	0.246	Assumed	50%	13.9
OUT07	91.4	91.4	100%	36.8	22515	0.26	0.26	100%	0.256	0.004	110	2%	11.4
OUT08	126.9	126.9	100%	64.2	37902	0.48	0.48	100%	0.347	0.133	2080	28%	9.1
TSOUT01A	NA	13.3	NA	67.2	3113	67.00	0.01	0.02%	0.006	0.006	Assumed	50%	1.9
TSOUT01B	NA	107.0	NA	70.6	4465	33.00	0.05	0.15%	0.024	0.024	Assumed	50%	5.4
TSOUT02	NA	3.3	NA	21.1	767	67.00	0.00	0.01%	0.002	0.002	Assumed	50%	2.2

NA: Not Applicable

Notes:

(1) Gross meter tributary area = sum of all sewered contributing areas upstream of a meter site

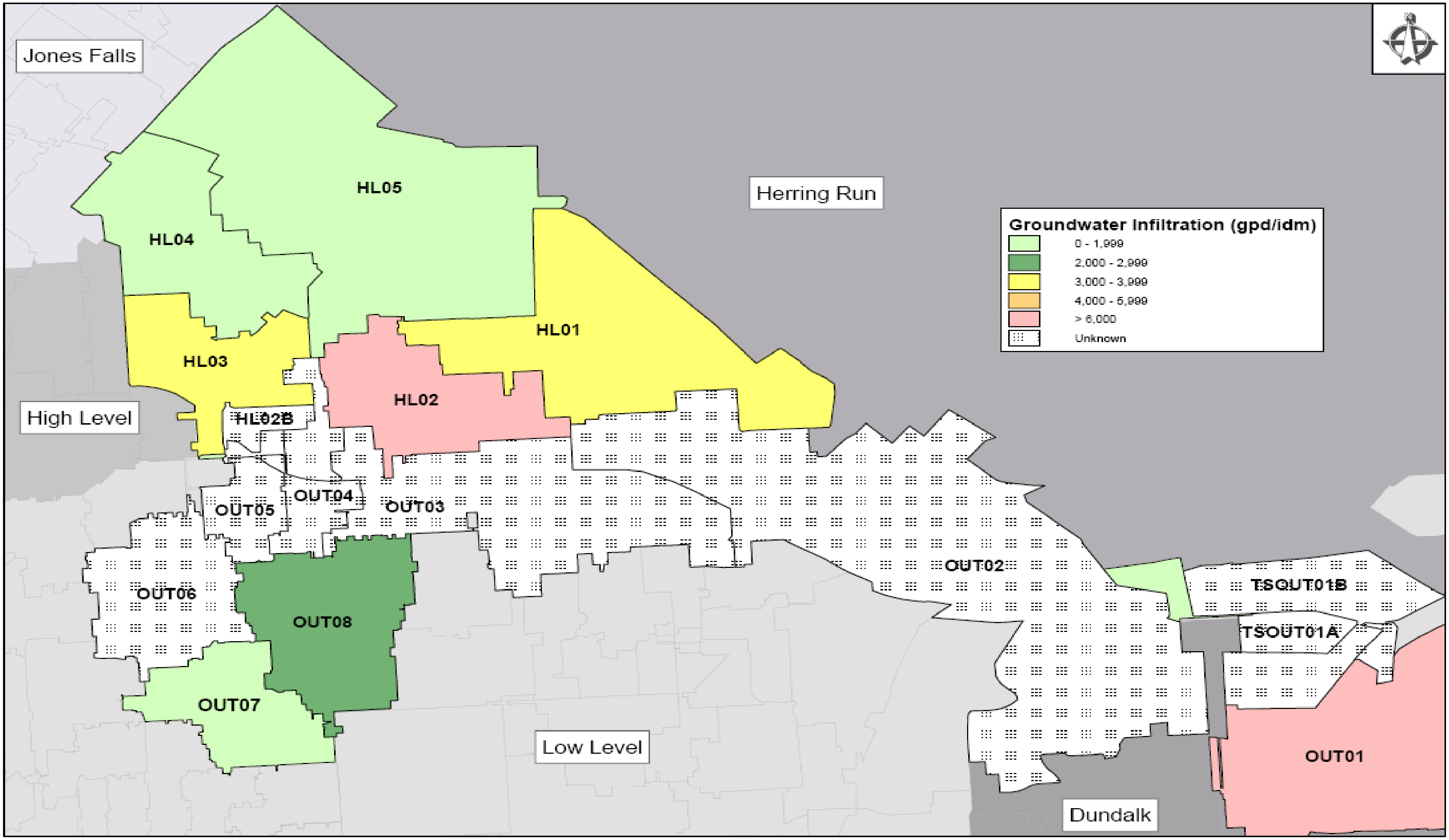
(2) Net meter basin area = incremental tributary area between meter sites

(3) Gross average daily flow at meter site for the gross meter tributary area

(4) Net average daily flow of the net meter basin area = $BSF_{net} + GWI_{net}$

(5) Net base sanitary flow (BSF) from the net meter basin area (called wastewater production (WWP) in Table 5 of the Jones Falls report)

(6) Net groundwater infiltration (GWI) (called base infiltration (BI) in Table 5 of the Jones Falls report)



Map 3.6.1: Outfall Base Infiltration Severity Map

3.6.2 Correlation with Completed CCTV and Manhole Inspections

For this correlation evaluation, the groundwater infiltration rates assigned to each meter basin were compared to the Manhole Inspection Data and the CCTV Sewer Inspection Data. Only the defects classified as infiltration defects were considered (not all types of defects).

Manhole Inspections: There does not appear to be a correlation between the groundwater infiltration rates assigned to the meter basins and the number of infiltration defects reported from the manhole inspection program. For example, meter basin HL04 has the lowest infiltration rate of the six basins evaluated, but has the highest number of manhole infiltration defects observed. Meter basin HL05 has the highest infiltration rate and is ranked 5th when considering the number of manhole infiltration defects observed.

CCTV Sewer Inspection: There also appears to be no correlation evident between the various infiltration defects observed in the CCTV sewer inspection data and the groundwater infiltration rates. Similar to the manhole defects described above, there is a lack of correlation between the infiltration rate and the infiltration defects observed in the CCTV data. For example, meter basin OUT08 has the 2nd highest infiltration rate, but has the lowest number of sewer infiltration defects in the CCTV database. Also, meter basin HL03 is ranked 4th in infiltration rate, but ranks as the basin with the highest number of observed sewer infiltration defects.

The lack of correlation between infiltration rates and observed defects in sewers and manholes is not unusual. Because privately owned sewer pipes were not inspected, it is not possible to draw conclusions about infiltration defects and possible sources of infiltration located in the private property sector.

3.6.3 Influence of Groundwater Table on Infiltration Rates

At the direction of BaSES Manual Section 7.4.5, the Base Infiltration Rates were separated and processed for each season for which there was data in the Sliicer database. While temperature and vegetation activity differences have an influence on soil moisture in these two typical seasons, these are not the only explanations for observing different R factors from event to event. A more likely and more significant cause is the influence of back-to-back storm events, which can elevate groundwater levels and submerge more sewer system defects and also inhibit the ground infiltration of rainfall and making more surface runoff available to enter the sewer system through defects such as missing cleanout caps and yard drains.

3.6.4 Base Infiltration from Baltimore County

As the Outfall Sewershed has no areas of Baltimore County contributing flow directly to it, there is no discussion of Base Infiltration from Baltimore County in this plan.

3.7 Wet Weather Analysis

The wet weather response of a meter basin to rainfall is quantified as the volume of I/I in proportion to the amount of rainfall. BaSES manual section 3.6.8 describes a variety of formats that are available in the Sliicer program to relate rainfall to I/I response. The "volume-to-volume" approach was used in this study. BaSES Manual section 7.4.6 describes the modeling approach to be used in the InfoWorks program to generate I/I flows in the hydraulic model for this study; it is the SWMM RUNOFF routine, which uses a capture coefficient (called the R-factor).

The wet weather response of meter basins in the Outfall Sewershed was initially evaluated in the Sliicer program. The initial parameters were used to start the model development and calibration in the InfoWorks hydraulic model. By comparing the hydraulic model simulations to the measured flows, refinements were made to the calibration parameters. The initial estimates of I/I response, using the Sliicer program to correlate I/I to rainfall, were based on the 30-minute averaged data in Sliicer. The refined estimates of I/I response using the InfoWorks model was based on the five-minute radar rainfall data (using the 1 sq km pixels).

A more complete analysis is included in the Attachment 3.8.1 – The Outfall Sewershed Inflow and Infiltration Report.

3.7.1 Observed Peak Flows

Peak flow data collected during the flow monitoring period at each meter site for the selected storm calibrations is shown in Appendix D of the Hydraulic Model Development and Calibration Report (Attachment 5.2.1).

3.7.2 RDII Rates and Severity

The rainfall dependent infiltration / inflow (RDII) is normalized so that the relative RDII rates of various meter basins can be compared fairly. Two approaches to normalization were used in this evaluation: a capture coefficient (R-factor) and a normalized RDII value. The R-factor is a deterministic, linear relationship between I/I and rainfall. It is a non-dimension ratio of the RDII volume to the rainfall volume. The normalized RDII value is the RDII volume divided by the length of pipe and the rainfall depth. For both approaches, the RDII volume is based on the "Storm Period Net RDII Volume" calculated by Sliicer. It is defined by the equation:

R-factor = (RDII volume $_{(MG)}$) / (Rainfall volume $_{(MG)}$)

As described in the BaSES Manual Section 3.6.8, a graphical technique for evaluating the performance of Sewershed basins under widely varying rain intensity is the "Q vs i" diagram. Q is the RDII volume and "i" is the corresponding rainfall depth. Appendix B contains plots of the "Q vs i" diagrams for each flow meter. The data points represent individual wet weather events, which form a scattered pattern when plotted.

The slope of a regression line on the "Q vs i" diagram is used in the following equation to obtain the capture coefficient:

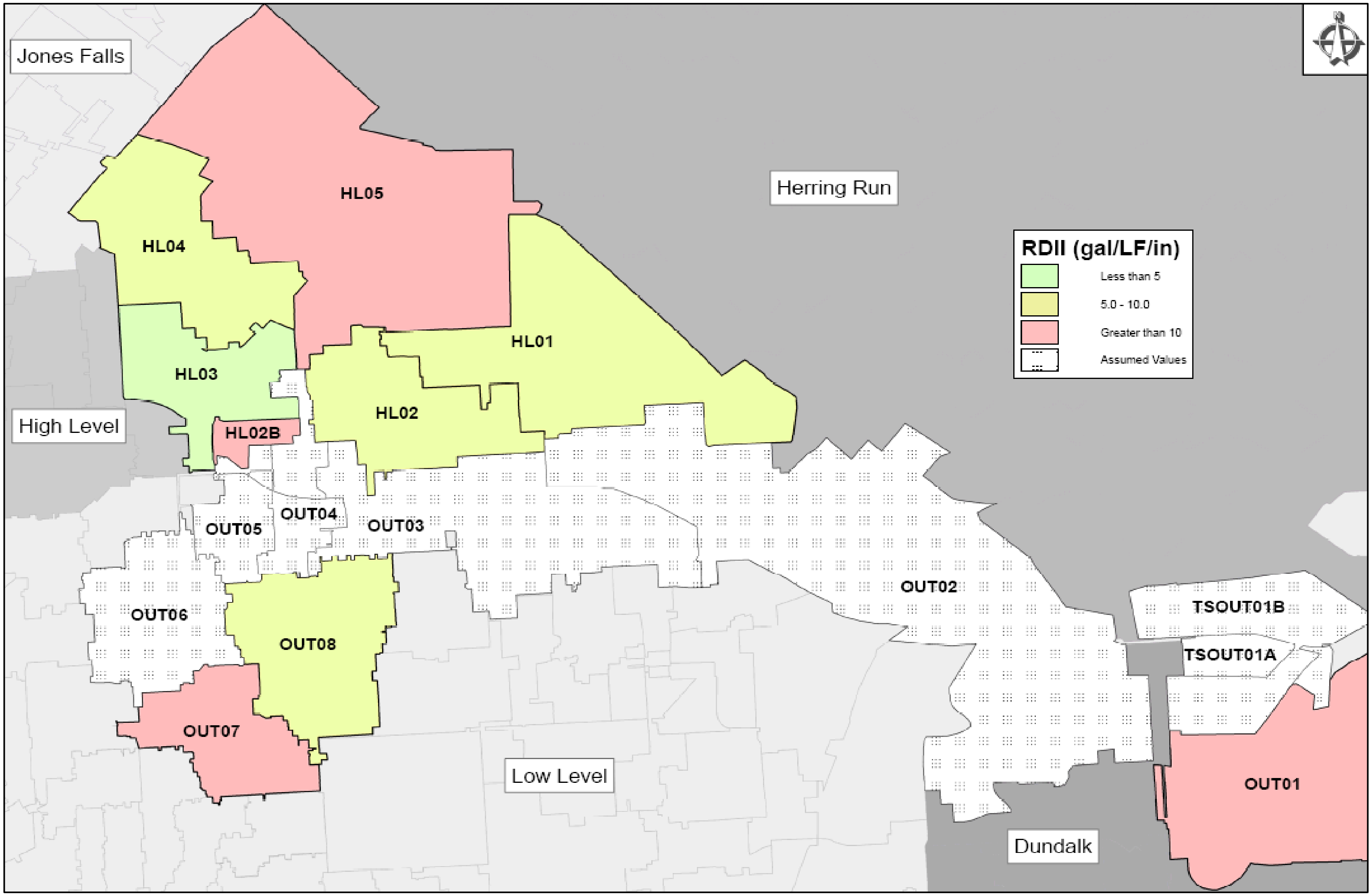
R-factor = $(36.8_{(\text{ac in/MG})}) \times (\text{Slope}_{(\text{MG/inch})}) / (\text{Area}_{(\text{acres})})$

The R-factor is used in the SWMM runoff method in the InfoWorks model to simulate the wet weather response. RDII was also normalized by length of piping and rainfall depth.

Meter basin RDII response was evaluated using "Q vs i" diagrams and scatter graphs of velocity and depth. The "Q vs i" diagrams are given in Appendix B and the results are summarized in Table 5-3 in which the meter basins are compared using RDII normalized by the linear feet of pipe and by meter basin area (R-factor method). Meter basins with RDII values greater than 10 gal/LF/in are: OUT01, OUT07, HL02B, and HL05. A similar, but not identical ranking of meter basins using R-factors identifies OUT07, HL02B, OUT01, and HL02 as the meter basins with the highest wet weather response. Meter basins with low wet weather response (by either normalization method) are: HL01, OUT08 and HL03.

Table 3.7.1 summarizes the model wet weather characteristics for the meter basins in the Outfall Sewershed. The table presents both the initial Sliicer analysis and the refined InfoWorks analysis that is discussed in Section 5. Map 3.7.1 depicts the RDII response normalized by pipe length for each meter basin.

Table 3.7.1 also ranks the meter basins from highest to lowest RDII response using both metrics. Using the refined calibration values, meter basins with RDII values greater than 10 gal/LF/in are: OUT01, OUT07, HL02B, and HL05. A similar, but not identical ranking of meter basins using R-factors identifies OUT07, HL02B, OUT01, and HL02 as the meter basins with the highest wet weather response. Meter basins with low wet weather response (by either metric) are: HL01, OUT08 and HL03. The meter basins with assumed I/I values are not included in the ranking.



Map 3.7.1: Outfall RDII Severity Map

Table 3.7.1: Wet-Weather Analysis Table

	Sliicer Analysis Preliminary Calibration		InfoWorks Analysis Refined Calibration	
Basin	**RDII (gal/LF/in)**	**Capture Coefficient R-factor**	**RDII (gal/LF/in)**	**Capture Coefficient R-factor**
HL01	10	3.6	6	2.0
HL02	10	9.4	8	8.0
HL02B	12	11.8	12	12.0
HL03	4	3.9	3	3.0
HL04	7	5.9	9	7.5
HL05	7	4.2	11	6.5
OUT01	21	10.0	21	10.0
OUT02	Main Trunk Sewer		44 Assumed	10 Assumed
OUT03	Main Trunk Sewer		15 Assumed	10 Assumed
OUT04	Main Trunk Sewer		15 Assumed	10 Assumed
OUT05	Main Trunk Sewer		10 Assumed	10 Assumed
OUT06	Main Trunk Sewer		15 Assumed	10 Assumed
OUT07	8	7.0	14	13.0
OUT08	4	4.4	5	5.0
TSOUT01A	Main Trunk Sewer		8 Assumed	6.5 Assumed
TSOUT01B	Main Trunk Sewer		65 Assumed	10 Assumed
TSOUT02	Main Trunk Sewer		12 Assumed	10 Assumed

	InfoWorks Analysis Refined Calibration	
Basin	**Ranking from Greatest to Least Based on RDII**	**Ranking from Greatest to Least Based on R-factor**
OUT01	1st	3rd
OUT07	2nd	1st
HL02B	3rd	2nd
HL05	4th	6th
HL04	5th	5th
HL02	6th	4th
HL01	7th	9th
OUT08	8th	7th
HL03	9th	8th

Scattergraphs also were evaluated to understand the state of flow, the influence of sediment, and whether the meter site is free flowing or influenced by backwater conditions. A review of the scattergraph plots included in Attachment 3.8.1 shows evidence of surcharging in many of the scatter graphs and some indication of potential sanitary sewer overflows (SSOs) in the Outfall Sewershed based on the flow meter data for OUT07 and OUT06A.

3.7.3 Correlation with Completed CCTV and Manhole Inspections

Similar to the correlation discussion presented for the Dry Weather Analysis (Section 3.6.2), there appears to be no distinct correlation between the assigned wet weather RDII values for each meter basin and the observed CCTV sewer defects and the manhole inspection results. It should be noted that the RDII values assigned to Outfall meter basins OUT02 through OUT06 were given arbitrary values of 10 because of the high upstream flows monitored relative to the small incremental flow generated in the local metershed area.

For the meter basins that were assigned a calculated R-value, there appears to be a somewhat tenuous correlation between the RDII and the manhole leaks reported from the manhole inspection program. For example, meter basin HL01 has the lowest RDII and a relatively low ranking of MH defects observed; and meter basin OUT07 has the highest RDII and a high number of MH defects observed (ranked 3rd). However, meter basin HL03 has the second lowest RDII, but the 2nd highest number of MH defects observed. Meter basin OUT08 has the third lowest RDII and the fourth highest MH defect number. This seems to indicate that there is no consistent relationship between the model parameters and the observed number infiltration defects.

There also appears to be no distinct correlation between infiltration observed in the CCTV inspections and the RDII assigned. Meter basin HL02B has the 2nd highest assigned RDII and the fourth lowest sewer defects found in the CCTV database. Meter basin HL01 has the lowest RDII and is ranked 10th with regard to sewer defects. Meter basin OUT07 has the highest RDII and ranks only 7th with regard to sewer defects observed. Meter basin OUT08 is ranked seventh in RDII and has the fourth highest number of sewer defects. No definite trend can be ascertained from the data evaluated.

The lack of correlation to the inspection results for the sewers and manholes is not unusual, and could be caused by potentially significant sources of RDII originating in the private property sector.

3.7.4 RDII from Baltimore County

As the Outfall Sewershed has no areas of Baltimore County contributing flow directly to it, there is no discussion of RDII from Baltimore County in this plan.

3.7.5 Smoke Testing Recommendations

The following meter basins were recognized to have the highest peak flow responses (observed in the Sliicer.com flow meter data) and were recommended for smoke testing to identify sources of inflow or rapid infiltration (listed in order of priority).

The meter basins are:

HL02
HL04
HL05
OUT07
OUT08
HL01

The meter basins with the greatest R-values are HL02B, OUT01 and OUT07; however, there are no smoke testing recommendation for HL02B and OUT01. HL02B was not recommended because it is a small basin with little flow; water depths are very shallow even during peak flow conditions. Basin OUT01 did not have data in the original data set used of prioritize the basins for smoke testing. However, subsequent data for OUT01 indicated that it would benefit from smoke testing. Smoke testing results are discussed in Section 4.4.

3.8 Outfall Sewershed Infiltration and Inflow Evaluation Report

Attachment 3.8.1 contains the Outfall Sewershed Inflow and Infiltration Report prepared by the Joint Venture Team. The report contains site reports, scattergraphs, hydrographs, and Q vs. I scattergraphs for every flow monitoring location.

4.0 Sewer System Evaluation Study

4.1 Overview

The Sewer System Evaluation Study consists of a wide range of activities as defined by the Consent Decree (CD). The primary assessment conducted for each of the City of Baltimore's sewersheds is important for characterizing the condition of the system as it provides important insight into the historical nature of the collection system. The testing and inspection of the wastewater collection system, in what is termed sewer system evaluation survey (SSES), is a significant part of the overall evaluation of the sewershed. These SSES activities include conducting flow monitoring and rainfall data collection programs, completing the inspection of manholes and other sewer structures located within the collection system, performing internal inspection of sewers 8-inches in diameter and larger, conducting smoke and dyed-water testing, the preparation, calibration and validation of a hydraulic model, the identification of critical sewer system components within the collection system, and establishing criticality ratings for these components. All data was compiled to formulate a long term rehabilitation and corrective action plan that includes an implementation schedule and estimates of probable costs.

The City provided guidance and general direction to the sewershed consultants to assure that all tasks completed in support of this study are prepared in a standardized format to facilitate the collection and review of the data for compliance with the requirements of the CD. The SSES emphasizes on the inspection of sanitary sewers 8-inches and larger in diameter, including all sewer structures per Paragraph 9 of the CD. This information will be used in the preparation of a comprehensive corrective action plan for the sewershed. As part of the Outfall Sewershed SSES, 327,771 linear feet (LF) of gravity sewer lines and approximately 2,195 manholes were inspected.

4.2 Manhole Inspections

Manholes are the principal means to access a collection system. As such, effective manhole inspection is important in characterizing the overall condition and connectivity of the collection system. The manhole inspections completed for this project typically served multiple roles, which included characterizing the condition of the structure, identifying system connectivity, assisting in defining the general condition of the sewer segments connected to the structure, providing defect observation data required for the condition assessment and development of subsequent repair recommendations for the structure, and identifying additional potential sources of Inflow and Infiltration (I/I) into the collection system. The inspections also provided updated system attribute data such as pipe diameters, structure type and depths, network connectivity, and sewer system configuration. Collection of this data during the detailed inspections also allowed the City's GIS to be updated accurately and efficiently. These updates included removing structures that were originally identified as sewer structures in the GIS system but were actually not, and accurately updating the GIS with newly identified sewers and sewer structures that were not originally shown in the GIS.

Manholes were inspected as required by the CD in accordance with general guidelines outlined in the Environmental Protection Agency's (EPA) SSES Handbook, the American Society of Civil Engineers (ASCE) Manhole Inspection and Rehabilitation Manual 92, and the newly defined requirements of the National Association of Sewer Service Companies (NASSCO) Manhole Assessment and Certification Program (MACP). All inspections were completed under the guidance of MACP certified inspectors. Manholes that could not be located or opened for inspection were documented for additional action. These structures will be inspected and incorporated into the City's overall rehabilitation plan.

Wherever physical manned-entry internal inspections were required, these were conducted in accordance with OSHA's 29 CFR 1910.146 Confined Space Entry Requirements.

Manhole inspections were conducted by Dewberry/Brown and Caldwell's sub-contractor, Phoenix Engineering, utilizing the Manhole Inspection Application Software (MIAS) Version 3.4 developed by the City of Baltimore DPW.

MIAS allowed field crews to collect detailed inspection information about the physical characteristics of a manhole or structure, identify any sewer connections to the structure and record details about the environment surrounding the manhole that was needed to accurately characterize the condition of the manhole or structure. In addition to the characteristics of the structure, such as the structure's size, shape and construction material, the MIAS application allowed defects and potential sources of I/I to be recorded. MIAS was designed to provide internal methods that link the inspection photographs of the manhole or defect observations to the manhole database record, making them available for easy review and preparation of formal reports to the City or for review at a later date. MIAS also allows access to the GIS and aerial maps, which provided the inspector with additional system or location information in the field to allow them to accurately complete the inspection and update the detailed inspection database.

The following is a brief description of the process involved in the collection of manhole inspection data for the Outfall Sewershed. The following descriptions are not intended to cover all aspects of the work performed, rather to provide the reader with a general understanding of the data collection and review process.

- A manhole inspection crew consisting of 2 inspectors uses a 1" = 100' scale GIS map to identify manholes to be inspected. This map contains information such as street names, manhole location and ID, flow direction and connectivity of the system with all other upstream and downstream manholes.

- The crew selects a manhole from the database list of manholes and goes to the location where the manhole is shown on the GIS map and performs a visual search in an effort to locate the manhole or structure for inspection. If found, the manhole is located utilizing GPS or other typical survey technique such as triangulation measurements, and then the manhole is inspected. If the manhole is not found, the position is estimated based on the surrounding objects shown on the map and methods such as probing the soil are used to try to locate the manhole for inspection.

- If a manhole structure is not found after a normal field investigation or cannot be opened, it is noted as "Cannot Locate (CNL)" or "Cannot Open (CNO)" in the MIAS database and sent to Dewberry/Brown & Caldwell's subcontractor REI Drayco for a specialized field investigation. Before the specialized field investigation is performed, CCTV records are checked to confirm that the manhole exists for CNL manholes. The specialized field investigation involves the use of a CCTV push camera with sonar signaling that is captured overland once the camera is inside a CNL manhole. If the manhole is covered by grass or pavement, the manhole is uncovered and the manhole height is adjusted. CNO manhole covers are often unbolted and lubricated to permit access by the inspection team. Once the manhole is made accessible, the inspection team is notified and they revisit the site and complete the inspection. If the manhole cannot be inspected because of impractical circumstances, it is placed on the city's Asset Accounting Database and the exact conditions of the failed inspection are documented for further evaluation under the City's overall rehabilitation plan.

- Once a manhole is located and opened, the MIAS survey is completed. The format of the MIAS inspection form prompts the inspector to begin their inspection by recording features such as the structure's location, then features and defects are recorded starting at the top of the manhole structure and working down to the invert. These entries include frame/cover type, condition, and materials of construction for the chimney, corbel, barrel, bench and channel and their current condition and evidence of I/I.

- Photographs are obtained and entered into the system for location views and top down views of the manhole; photographs are also collected for the pipe connections and any significant defects when possible.

- Pipe sizes are recorded and located according to clock position with the outgoing pipe always being the 12 o'clock position. Pipe diameter and rim to invert depths are also collected and recorded in MIAS along with the condition of the pipe seals.

- All manholes are then assigned a 1-5 condition rating, with 1 being in excellent condition and 5 being in very poor condition and requiring immediate attention.

In order to prioritize the maintenance and repair of the manholes, a condition rating scale was used to weight the various types of structural defects and I/I conditions that occurred in different components of the manhole structure. This rating system also allowed for the characterization of operation and maintenance (O&M) type issues such as identification of fats, oils and grease (FOG), debris accumulations, surcharging of the manhole and other O&M type issues. During the initial phase of this project, NASSCO introduced a standard for manhole condition assessment. This standard was the Manhole Assessment and Certification Program (MACP), which was subsequently adopted by the City to aid in the consistency of data collected and to provide for a reliable evaluation of each manhole component. The use of this standard provides a baseline condition assessment of the structure, which aids in providing a consistent review of conditions during future inspections. The 1-5 condition rating standard used for the manhole inspections is largely based on the ASCE Manual of Practice No. 92, which utilizes a 5-point severity rating system. The following represents the rating scale:

1. Excellent Condition – Only minor defects
2. Good Condition – Defects have not started to deteriorate
3. Fair Condition – Moderate defects that will continue to deteriorate
4. Poor Condition – Severe defects likely to become a grade 5
5. Immediate Attention Required – Defects requiring immediate attention

Table 4.2.1 and 4.2.2 provides an overview of the condition of the 2,195 manholes inspected as part of the Outfall Sewershed manhole inspection program and classifies the manholes by overall structure rating. The manhole condition rating of the 47 manholes associated with the Outfall Interceptor, Outfall Relief and 99-inch Sewers are presented separately from the rest of the manhole ratings in Table 4.2.2.

It should be noted, that not all the manhole inspections associated with the Outfall Interceptor, the Outfall Relief, and 99-Inch Sewers have been completed at the time of this draft. Manholes that are accessible are anticipated to be completed by the end of February, 2010, and the results, noted deficiencies, and recommended improvements will be incorporated into the next draft submittal.

Table 4.2.1 - Manhole Condition Rating Summary - On Sewers < 99"		
Rating	**Count**	**%**
5 - Defects that require immediate attention	2	0.1%
4 - (Poor)	13	0.7%
3 - (Fair)	1,586	80.8%
2 - (Good)	46	2.3%
1 - (Excellent)	0	0.0%
*Missing Rating**	316	16.1%
TOTAL MANHOLES INSPECTED	**1,963**	**100%**

Table 4.2.2 - Manhole Condition Rating Summary - On Sewers 99" & Larger		
Rating	**Count**	**%**
5 - Defects that require immediate attention	0	0.0%
4 - (Poor)	0	0.0%
3 - (Fair)	31	53.4%
2 - (Good)	0	0.0%
1 - (Excellent)	0	0.0%
*Missing Rating**	27	46.6%
TOTAL MANHOLES INSPECTED	**58**	**100%**

** These are the manholes that could not be located or could not be opened for inspection.*

Table 4.2.3 provides an overview of the general manholes defect quantities within the 1,963 manholes located on sewers less than 99-inches in diameter.

Table 4.2.3 - General Manhole Defect Summary - On Sewers < 99"		
Description	**Count**	**%**
Frame leaks	1,005	51.2%
Chimney leaks	1,545	78.7%
Corbel leaks	1,513	77.1%
Barrel leaks	1,508	76.8%
Bench leaks	12	0.6%
Channel leaks	4	0.2%

Table 4.2.4 provides an overview of the general manholes defect quantities within the 58 manholes located on sewers with diameters 99-inches and larger.

Table 4.2.4 - General Manhole Defect Summary - On Sewers 99" & Larger		
Description	**Count**	**%**
Frame leaks	26	44.8%
Chimney leaks	16	27.6%
Corbel leaks	28	48.3%
Barrel leaks	26	44.8%
Bench leaks	0	0.0%
Channel leaks	0	0.0%

Table 4.2.5 provides an overview of the total number of defects observed, classifying the conditions by defect type within the 1,963 manholes located on sewers less than

99-inches in diameter. Attachment 4.2.1 contains all manhole inspection reports completed for this project.

Table 4.2.5 - Manhole Defect Location Summary - On Sewers < 99"		
Description	**Count**	**%**
MH Cover Defects	9	0.2%
MH Frame Defects	1,059	20.2%
MH Chimney Defects	1,051	20.1%
MH Corbel Defects	73	1.4%
MH Barrel Defects	66	1.3%
MH Bench Defects	937	17.9%
MH Channel Defects	683	13.1%
MH Steps	1,353	25.9%
Total:	**5,231**	**100%**

Table 4.2.6 provides an overview of the total number of defects observed, classifying the conditions by defect type within the 58 manholes located on sewers with diameter 99-inches and larger.

Table 4.2.6 - Manhole Defect Location Summary - On Sewers 99" & Larger		
Description	**Count**	**%**
MH Cover Defects	1	0.9%
MH Frame Defects	28	25.9%
MH Chimney Defects	6	5.6%
MH Corbel Defects	2	1.9%
MH Barrel Defects	9	8.3%
MH Bench Defects	30	27.8%
MH Channel Defects	5	4.6%
MH Steps	27	25.0%
Total:	**108**	**100%**

4.3 Sewer Cleaning and Closed Circuit Television Inspection (CCTV)

Internal inspection of sewer pipes is the process of inspecting and documenting the condition of the pipes by means of CCTV. It also provides valuable insight into the cleaning and maintenance requirements of each sewer segment and provide information that is needed to assign appropriate rehabilitation technologies to deteriorated or damaged pipe segments.

To provide the highest visibility of defects, all sewers inspected were cleaned prior to inspection to accurately define the conditions. Sewers were cleaned utilizing hydraulically propelled high-velocity jet or other mechanically powered equipment. The intent of the cleaning operations was twofold. First, to adequately clean the sewer so the inspection could identify defects that otherwise would not be visible and second, to remove all foreign materials from the sewer to restore the sewer to a minimum of 95% of its original carrying capacity. When significant restrictions such as roots or other heavy debris restrictions were encountered, heavy cleaning was utilized to restore the capacity of the sewers and allow for internal inspection. Heavy cleaning involved root cutting or additional passes of the hydro-cleaning equipment. All debris was removed from the sewers. When significant blockages were identified, they were reported to the City and the City promptly coordinated with the wastewater maintenance division or their on-call contractor to resolve the deficiencies.

Following cleaning, the sewer segments were inspected by means of CCTV. These inspections were used to identify the following:

- Current pipe condition including existing or potential structural deficiencies or problems, and accurately identifying the pipe's connectivity and location.
- Confirmation, extent and current condition of previous rehabilitation projects and/or repairs.
- Identifying improper or potentially illicit connections.
- Identifying potential sources of I/I.
- Assist in selecting appropriate methods of repair, rehabilitation and/or replacement.

Paragraph 9 of the CD requires that gravity sewers eight (8) inches and larger in diameter be inspected using CCTV inspection in accordance with NASSCO guidelines. The CCTV inspection of the sewers provided the necessary condition assessment for the SSES evaluation of the Outfall Sewershed. The inspections identified defects and other problems relating to the sanitary sewer collection and conveyance system that allows the project team to compile a comprehensive corrective action plan and prioritize an implementation schedule.

All CCTV inspections were completed and data collected according to NASSCO's Pipeline Assessment and Certification Program (PACP) guidelines and standards. The City required the use of PACP certified software to collect and record all CCTV information. All CCTV operators, equipment and the review team were certified in the use of the PACP coding system.

All CCTV inspections were conducted using a color pan-and-tilt, radial viewing inspection camera that provides adequate illumination to clearly observe defects and other features within the pipe. All surveys were initiated from the upstream manhole proceeding downstream with the flow to minimize splashing of the camera. When defects or other obstructions prevented the completion of the inspection in this direction, a reverse inspection was initiated from the downstream manhole to complete the inspection of the sewer segment. The CCTV camera lens was required to be positioned in the center of the pipe being inspected and movement of the camera through the sewer pipe did not exceed a speed of 30-feet per minute. Wastewater flows in the sewer during the inspection were controlled and did not exceed 20 percent of the pipe capacity for pipes 8”- 10”; 25 percent for pipes 12”- 24”, and 30 percent for pipes 24” and larger per the PACP guidelines. During the internal inspection, the CCTV camera was temporarily stopped at all significant defects and side sewer or service connections to accurately code and provide a clear image of the defect or point of connection. For larger sewer inspections where it was not practical or when flows could not safely or effectively be reduced, sonar inspection or a combination of sonar and CCTV inspection was used to inspect the sewers. The use of a combination CCTV/sonar camera allowed for the visual inspection of the sewer above the flow line and the sonar provided inspection information below the flow of the sewer.

As a means to prioritize the maintenance and repair of pipe sections and other associated sewer appurtenances, a condition rating scale was used to rate the various types and degrees of structural defects and I/I conditions occurring in different segments of the sanitary sewer system. The PACP rating scale was utilized as a standard and consistent format for the way pipes were evaluated and conditions recorded. These standards allow pipe conditions to be reported in a standard recognized manner and allow the City to compare the segment's condition from one time frame to another and accurately track the condition of the pipe and any progression of defects.

The PACP coding system requires the assignment of a specific code for each structural and O&M type defect identified within a pipe segment. The software automatically assigns a PACP rating code to each defect when entered. These grades are assigned based on the potential for further deterioration or possible failure of the pipe.

The PACP grading system obtained from NASSCO's "Pipeline Assessment and Certification Program" reference manual utilized for this project is as follows:

Grade	Description	Time to Failure
5	Immediate Attention Required	Pipe has failed or will fail within 5 years
4	Poor	Pipe will probably fail within 5 to 10 years
3	Fair	Pipe may fail in 10 to 20 years
2	Good	Pipe unlikely to fail for at least 20 years
1	Excellent	Failure unlikely in the foreseeable future

Utilizing this system, each pre-defined defect or observation code is directly associated with a severity rating based on the type and extent of the defect. These ratings aid in determining the need for maintenance, repair, rehabilitation or replacement of the pipe segment. The PACP software assigns a four-digit severity code, or PACP quick rating for each sewer segment inspected and contained in the database. These ratings, in conjunction with the critically rating of the system component were what were used to prioritize system repairs.

Tables 4.3.1 and 4.3.2 summarize the defects recorded during the CCTV inspections by type of defect and also by overall segment condition rating. Table 4.3.3 summarizes the O&M conditions. Attachment 4.3.1 is an Access database that contains all CCTV inspection information completed as part of the CCTV inspection program in the Outfall Sewershed.

Table 4.3.1 - CCTV Defect Observation Summary

CCTV Inspection Defects		Pipe Diameter						
Family	Group Type	8" - 12"	14" - 18"	20" - 33"	36" - 56"	60" - 96"	>96"	Total
Structural	Broken or Hole	621	5	7	0	0	1	634
Structural	Collapse	8	0	0	0	0	0	8
Structural	Cracks	5,048	87	129	0	0	49	5,313
Structural	Defective Joints	570	0	2	0	0	0	572
Structural	Defective Lining	39	0	0	0	0	12	51
Structural	Deformation	50	0	0	0	0	0	50
Structural	Encrustation & Scale	120	1	0	0	4	343	468
Structural	Fracture	3,933	44	67	0	0	6	4,050
Structural	Repair	112	1	0	0	0	0	113
O&M	Encrustation & Scale	1,804	53	66	0	0	159	2,082
O&M	Grease	1,362	18	31	0	0	0	1,411
O&M	Infiltration	176	30	10	0	0	118	334
O&M	Obstruction	677	23	20	0	0	5	725
O&M	Roots	2,362	38	26	0	0	4	2,430
O&M	Settled Deposits	1,321	45	25	0	2	180	1,573
Constructional	Defective Tap	1,563	29	17	0	0	8	1,617
Constructional	Line Deviations	179	2	3	0	0	57	241
Constructional	Obstruction	131	0	3	0	0	9	143
Misc	Camera Underwater	38	2	0	0	0	4	44
Misc	Survey Abandoned	449	12	6	0	1	3	471
Misc	Water Level >20%	1,058	24	21	0	1	95	1,199
	Total:	**21,621**	**414**	**433**	**0**	**8**	**1,053**	**23,529**
	Percent:	**91.9%**	**1.8%**	**1.8%**	**0.0%**	**0.0%**	**4.5%**	**100%**

Table 4.3.2 - Sewer Overall Condition Rating Summary - For Sewers < 99"

Rating	Pipe Segments		Pipe Lengths	
	Count	%	Feet	%
5 - Defects that require immediate attention	11	0.6%	2089	0.7%
4 - (Poor)	8	0.4%	1,975	0.6%
3 - (Fair)	17	0.9%	4,185	1.4%
2 - (Good)	85	4.4%	19,292	6.3%
1 - (Excellent)	1,426	73.2%	229,229	75.4%
Missing pipes	401	20.6%	47,215	15.5%
Total:	**1,948**	**100%**	**303,985**	**100%**

Table 4.3.3 - Sewer Operation and Maintenance Condition Rating Summary - For Sewers < 99"				
Rating	Pipe Segments		Pipe Lengths	
	Count	%	Feet	%
5 - Defects that require immediate attention	9	0.5%	2,397	0.8%
4 - (Poor)	9	0.5%	2,228	0.7%
3 - (Fair)	51	2.6%	13,644	4.5%
2 - (Good)	196	10.1%	40,752	13.4%
1 - (Excellent)	1,282	65.8%	197,750	65.1%
Missing pipes	401	20.6%	47,215	15.5%
Total:	**1,948**	**100%**	**303,985**	**100%**

Closed Circuit Television (CCTV) / Sonar Inspections

For the Outfall Interceptor, Outfall Relief, and 99-inch Sewers, flows could not be effectively reduced to allow for inspection by CCTV Camera only. Thus, the Outfall Interceptor, Outfall Relief, and 99-inch sewers were inspected by use of a combination CCTV / Sonar Camera. The CCTV allowed for visual inspection of the sewers above the flow line, and the sonar provided inspection information below the flow line.

The CCTV inspections revealed exposed aggregate along the sidewall of all three sewers, and areas of missing aggregate along the crown of the pipe and along the flowline of pipe of all three sewers. The Outfall Interceptor and the 99-inch Sewers, with some minor exceptions, are unreinforced concrete structures. Without steel reinforcement in the pipe to act a point of reference, it was not possible to accurately determine from CCTV Tapes the extent of wall material loss. This was also an issue with the Outfall Relief Sewer.

Table 4.3.1 summarizes the defects, for all pipe sizes, recorded during the CCTV inspections by type of defect. Table 4.3.4 summarizes the defects, in the Outfall Interceptor, Outfall Relief and 99-inch sewers, by overall segment condition rating. Table 4.3.5 summarizes the O&M conditions for the three large diameter sewers.

Table 4.3.4 - Sewer Overall Condition Rating Summary - For Sewers 99" & Larger				
Rating	Pipe Segments		Pipe Lengths	
	Defect Count	%	Feet	%
5 - Defects that require immediate attention	5	4.4%	1,439	5.7%
4 - (Poor)	2	1.8%	663	2.6%
3 - (Fair)	100	87.7%	22,888	90.6%
2 - (Good)	1	0.9%	2	0.0%
1 - (Excellent)	6	5.3%	275	1.1%
Total:	**114**	**100%**	**25,267**	**100%**

Table 4.3.5 - Sewer Operation and Maintenance Condition Rating Summary - For Sewers 99" & Larger				
Rating	**Pipe Segments**		**Pipe Lengths**	
	Defect Count	**%**	**Feet**	**%**
5 - Defects that require immediate attention	0	0%	0	0.0%
4 - (Poor)	36	31.6%	8,765	34.7%
3 - (Fair)	73	64.0%	15,309	60.6%
2 - (Good)	0	0%	0	0.0%
1 - (Excellent)	5	4.4%	1,193	4.7%
Total:	**114**	**100%**	**25,267**	**100%**

Due to amount of flow in the large sewers, Sonar inspection below the flow line was conducted in conjunction with the CCTV inspection. Sonar inspection revealed accumulation of debris in the large sewers.

- Sediment / debris build-up in the Outfall Interceptor ranges from 21 inches up to 42 inches along the entire length.
- Sediment / debris build-up in the Outfall Relief Sewer ranges from 21 inches up to 41 inches. In one 400 foot segment, half of the pipe is filled with sediment / debris.
- Sediment / debris build-up in the 99-inch Sewer ranges from 10 inches up to 21 inches along the entire length.

4.4 Smoke Testing

Smoke testing was utilized by the project team as a means to quickly and effectively identify potential locations of stormwater/groundwater entry into the sanitary sewer collection system. Direct connections including downspouts, area drains, driveway drains, stairwell drains, patio drains, and storm sewer inlets or ditches can be confirmed with smoke testing. Indirect connections from storm sewers or drainage ditches, which allows I/I to pass through soil and into deteriorated or damaged conveyance piping, can also be identified with smoke testing.

Map 4.4.1 shows the meter basins that were smoke tested and the ones that were not. The smoke testing operations for this project were conducted between June and July 2008, during periods when the groundwater table was low and with sufficient time having elapsed from any prior rain events. Smoke testing was not allowed to be completed until 24-hours had passed from a wet-weather event to make sure the soils were sufficiently dry to allow detection of smoke. Prior to initiating the smoke testing, an extensive list of property owners, hospitals, nursing homes, schools, daycares, local civic and community leaders, community associations, council members, and police and fire officials were notified. This process included monthly testing notifications and the distribution of detailed smoke testing door hanger notifications, typically extending two blocks outside the test areas three days prior to conducting the tests. When smoke testing was initiated and subsequently stopped because of rain, re-initiation of the testing did not occur until conditions were again suitable and the notification process was completed again. In most cases smoke testing was conducted using a single blower setup technique with smoke being introduced at the smoke blower and pushed through isolated sections of the pipe. The maximum allowable set-up length was no more than two total manhole reaches. A manhole reach is defined as a manhole to manhole segment of the sewer. Field crews were responsible for determining that adequate smoke coverage was obtained by observing smoke concentrations and observing smoke travel using house plumbing vents along the setup. Smoke was continually introduced into the test setup manhole until adequate smoke coverage was obtained in the test area. In the event that smoke did not travel the entire reach, the setup was reversed by setting the blower on the opposing manhole of the initial setup and re-introducing the smoke. Such situations were often caused by pipe sags that contained flow, grease, debris, collapsed pipes, or other obstructions that would prevent smoke from traveling through the pipe. All instances were documented as a potential maintenance problem and reported to the City.

Both the upstream and downstream manholes were isolated during the smoke testing to concentrate the smoke within the test section. These restrictions were accomplished using sandbags or air plugs. In situations where heavier smoke concentrations were required, a dual blower technique was utilized with a blower placed on both the upstream and downstream manholes and smoke generated at each blower setup. The maximum set-up length in this situation was typically limited to 1,000 LF. Suspect inflow sources such as driveway drains, stairwell drains, window well drains, patio and area drains, and downspouts piped underground, or foundation drains were noted. Significant potential sources of “clear water” connections (such as storm drain or catch basin connections) were noted and were recommended for follow-up dyed-water testing to determine if actual cross connections existed. Care was taken to inspect the property around all buildings for sources of smoke. In situations where heavy smoke exited a source and it could be determined and documented through observation that the source was directly connected to the sanitary sewer, further investigation was not necessary. Generally, in all other situations where it could not accurately be determined if the source was directly connected to the sanitary sewer, further dyed-water testing was scheduled.

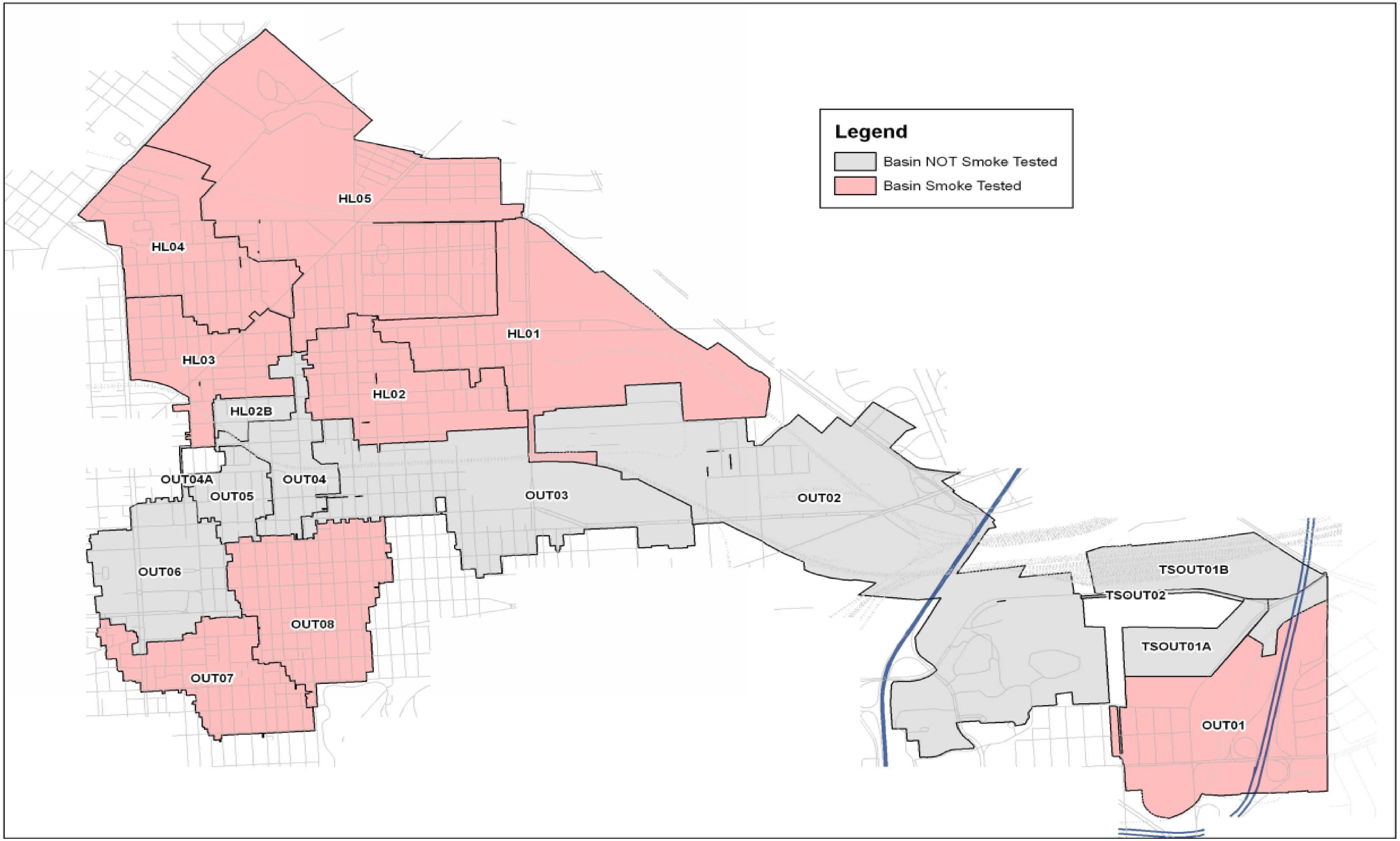
Table 4.4.1 summarizes the defects identified during the smoke testing inspections, identified by type of defect or source, defect location, sector (public or private) and the severity of the defect. Attachment 4.4.1 contains all smoke testing inspection data completed for this project.

Table 4.4.1 - Smoke Testing Defect Summary

Sector	**Total Defects**	**Percent Total (%)**
Public	160	21.78%
Private	555	78.22%
Total:	**715**	**100%**

Source Type	**Sub-Sewersheds**											
	HL01	**HL02**	**HL03**	**HL04**	**HL05**	**OUT01**	**OUT02**	**OUT03**	**OUT07**	**OUT08**	**No Basin**	**Total Observations**	**Percent Total (%)**
Building - Interior	1				1				1			3	0.42%
Cleanout	24	23	7	126	47	79			10	10	9	335	46.85%
Downspout		2			2							4	0.56%
Foundation Drain	1											1	0.14%
Lateral	4	10	5	48	61	39	1	1	9	20	5	203	28.39%
Lateral/Cleanout					3							3	0.42%
Main Sewer	1	2	3	35	27		1		5	1	4	79	11.05%
Main Sewer/Lateral				1								1	0.14%
Manhole Frame	6	3		4	3					2		18	2.52%
Other	12	8		2	4	1						27	3.78%
Storm Drain	3	7		2	3	4		1		4		24	3.36%
Storm Manhole		1				1						2	0.28%
Telephone Pole		1		4	3		1				2	11	1.54%
Water Line	1			1						2		4	0.56%
Total:	**53**	**57**	**15**	**223**	**154**	**124**	**3**	**2**	**25**	**39**	**20**	**715**	**100%**

Defect items in Table 4.4.1 that were coded as 02 – Service Connections, 03 – Cleanouts, 04 – Downspouts, 05 – Area Drains, 06 – Driveway Drains, 07 – Stairwell Drains, 08 – Foundation Drains, 09 – Building Interior, 11 – Storm Drain, 12 – Catch Basin/inlet, 13 Storm Manhole and 14 – Storm Ditch were scheduled for additional investigation utilizing dyed-water testing.



Map 4.4.1: Smoke Tested Basins

4.5 Dyed-Water Testing

The dyed-water testing or flooding of areas identified in Table 4.4.1 such as storm drain catch basins were conducted as part of the study of the Outfall sewershed collection system. The dyed-water flood test aided the project team in detecting pipe segments that were either direct or indirect connections between the storm drain and sanitary sewer system. Direct connections were typically confirmed during the smoke testing operations; however, any suspect locations were further investigated using dyed-water flooding or tracing. To complete this testing, the suspect storm drain, catch basin or other area was flooded with dyed-water and the adjacent connecting sanitary sewer manholes were observed for the presence of dye in the flow. In more detailed situations, the storm drain was plugged and filled with dyed-water and allowed to sit for an extended period of time to allow the dyed-water to permeate the surrounding soils and identify leakage points in the collection system piping. Typically a waiting period of at least thirty minutes following the initiation of the dyed-water was used before the test could be considered negative. The CCTV survey records performed in the vicinity of the defects were reviewed to determine if the defects were illegal connections. The existence of illegal connections was not observed in the CCTV surveys and the defects are considered to be indirect connections.

Table 4.5.1 summarizes the tests by location and type, and identifies all locations where dyed-water was observed through defects during the dyed-water testing inspections. Attachment 4.5.1 contains all dyed-water tests completed as part of this project.

Table 4.5.1 - Dyed-Water Testing Defect Summary

Sector	Count	Percent
Public	20	100.0%
Private	0	0.0%
Total	**20**	**100%**
Source	**Count**	**Percent**
Mainline	13	65.0%
Service Line	5	25.0%
Cleanout	0	0.0%
Downspout	0	0.0%
Area Drain	0	0.0%
Driveway Drain	0	0.0%
Stairwell Drain	0	0.0%
Foundation Drain	0	0.0%
Building Interior	0	0.0%
MH Frame/Seal	2	10.0%
Storm Drain	0	0.0%
Catch Basin/Inlet	0	0.0%
Storm Manhole	0	0.0%
Storm Ditch	0	0.0%
Excavation	0	0.0%
Other	0	0.0%
Total	**20**	**100%**

Note: Does not include Dyed-Water Testing results from OUT01 meter basin

4.6 Emergency Repairs / Rehabilitation

In accordance with Paragraph 9 Item C.iii of the Consent Decree, all significant system deficiencies observed during field inspections or when reviewing the field data were reported to the City. Figure 4.6.1 indicates the location of reported deficiencies and includes those discovered by everyday City operations.

4.7 Pumping Station Evaluations

There are no pump stations within the Outfall sewershed.

4.8 Quality Assurance / Quality Control Procedures

The following sections provide the reader with a brief description of the Quality Assurance / Quality Control (QA/QC) review process that all inspections underwent before they were considered complete and delivered to the City. In addition, copies of the Manhole Condition Rating and Defect Manuals, CCTV Review Manual and Smoke and Dyed-Water Testing Procedures Manuals developed by RK&K to insure the consistency and accuracy of the data being provided to the City are included as Attachments 4.8.1 through 4.8.4 of this report.

4.8.1 Manhole Inspection QA / QC Procedures:

- MIAS contains several internal field checks, which prompt the inspector to verify information as it is entered. (e.g.: if an inspector enters the invert elevation of an outgoing pipe at a higher elevation than the incoming pipe's invert elevation, the check prompts the inspector to verify the information). Several of these internal checks will not allow the inspector to move on to the next entry item in the inspection until the prior inspection item has been successfully completed.

- Basic information regarding location and system connectivity was compared with existing information or contract documents. Connecting manhole nodes entered in MIAS were compared to what was shown on the mapping and corrections made as necessary.

- All information was reviewed, which included reviewing for errors, assuring photograph quality and reviewing all comments entered by the inspector for clarity and content.

- If there was information missing, the MIAS record was failed and returned to a field crew to revisit the site and collect the required information or the reviewer would utilized existing record documents to obtain the required information.

- When the follow-up information was collected by the field crew or addressed by the reviewer utilizing record data, the new information was again reviewed and if acceptable, added to the record. The record was then tagged as QA/QC complete and flagged for submittal to the City.

4.8.2 CCTV Inspection QA / QC Procedures:

- All CCTV inspections were reviewed for conformance with PACP coding guidelines (video quality, flow levels, header information, all defects coded, and coded properly).

- Review all CCTV footage and inspection logs for significant defects such as collapsed pipe, blockages, etc. and forwarded these defects to the City for action.

- Review CCTV footage and inspection logs for significant O&M items such as excessive grease, roots, etc. and forwarded these defects to the City for action.

- If issues were found with video quality or PACP coding of defects for the segment inspected, the inspection record was returned to the CCTV contractor with review comments for recoding or re-surveying.

4.8.3 Smoke Testing QA / QC Procedures:

- All completed field reports were reviewed for conformance to the project guidelines and accuracy assuring that all maps, defect information and photographs are complete, clear, accurate and compatible.

- Review all smoke testing entries entered into the Access database to assure all observations and photographs are in accordance with the database scheme and specifications outlined for the project.

- If any field data collected was questionable, incomplete or illegible, the data was returned to the responsible contractor with review comments for correction and resubmission.

- Review all data submitted to identify significant defects such as cross connections. Any significant findings were reviewed and if required, assigned for further evaluation utilizing dyed-water testing.

4.8.4 Dyed-Water Testing QA / QC Procedures:

- All completed field data was reviewed for conformance with the project guidelines and accuracy requirements assuring that all maps, defect information and photographs are complete, clear, accurate and compatible.

- All dyed-water testing information was entered into the Access database to assure all observations and photographs are in accordance with the database scheme and specifications outlined for the project.

- If any field data collected was questionable, incomplete or illegible, the data was returned to the responsible contractor with review comments for correction and resubmission.

5.0 Hydraulic Modeling

Paragraph 12 of the CD defines the requirements of the collection and transmission system model. Consent Decree paragraph 12E requires a certification that the sewershed model includes the elements required in paragraphs 12A and B. The model is capable of and can be used for predicting the volume of wastewater flow, the hydraulic grade line (water levels) at any point in the modeled system, the capacity of the system, and the locations where overflows are likely. The model configuration is based on representative, accurate, and verified system attribute data. The model has been calibrated and validated with spatially and temporally representative rainfall and flow data collected during the flow monitoring program.

Modeling requirements per the Consent Decree are defined in even greater detail in the BaSES Manual, Section 7 (Hydraulic Modeling). The appropriate sections of the BaSES manual will be cited throughout this report to clearly identify how the development, calibration, and application of the Outfall Sewershed model fulfills the requirement of the BaSES manual and the objectives of the Consent Decree.

Following the guidance of Paragraph 12.B of the CD, the Outfall Sewershed model is capable of predicting:

- The volume of wastewater flow in the major gravity lines,
- Hydraulic pressure or hydraulic grade line of wastewater at any point in the major gravity lines,
- Likelihood and location of overflows under high flow conditions and considering normal in-line storage capacity.

The model is also:

- Configured based on representative, accurate, and verified system attribute data (i.e., pipe sizes and invert elevations, manhole rim elevations, etc.),
- Calibrated using spatially and temporally representative rainfall data and flow data obtained during the rainfall and flow monitoring, and
- Verified using spatially and temporally representative rainfall data and flow data; that data shall be independent of the data used to calibrate the model.

5.1 Model Network

In general, a hydraulic model contains three essential components:

- Network of sewer infrastructure (pipes, pumps and structures);
- Tributary basins served by the sewer network (i.e., the source of flows to the network), and

- Boundary conditions (i.e., upstream inflows and downstream water levels that represent the system beyond the model boundaries).

Figure 5.1 is a schematic of the large diameter trunk sewers in the Outfall Sewershed. The schematic shows the points of inflow to the Outfall Sewershed model from the upstream sewersheds and the locations of the downstream boundary conditions at the County Line.

Approximate capacities of the trunk sewers are noted on the schematic for a clean condition if sediment were removed and for the existing condition with sediment. The capacities are given as ranges to account for the variable depth of sedimentation in the existing condition and for a possible range of pipe roughness values in the clean condition (Manning's roughness from 0.015 to 0.013). The representative inflow rates noted on the schematic are approximate values for the typical inflows from upstream sewersheds in a large wet weather event; these values are for conceptual reference. Inflow hydrographs provided by the technical program manager were used for the model simulations.

The Joint Venture Team used the InfoWorks™ CS hydraulic modeling software by Wallingford Software to build a hydraulic network model of the Outfall Sewershed. The InfoWorks™ model satisfies the requirements of Consent Decree paragraph 12B and is useful to perform a dynamic hydraulic evaluation of the sewer system in accordance with Paragraph 9.F of the Consent Decree.

Consent decree paragraph 9.F(i) gives general instructions for the model development, which are specified in greater detail in BaSES 7.4.1. The model network contains:

- All gravity lines that are 10-inches in diameter or larger
- All 8-inch sewer lines that convey or are necessary to accurately represent flow attributable to a service area in each of the collection system sewershed service areas
- All gravity sewer lines that convey wastewater from one pumping station service area to another pumping station service area
- All gravity sewer lines that have caused or contributed to, or that the City knows are likely to cause or contribute to capacity-related overflows
- All manholes, junctions, and structures along modeled sewer lines
- Simulated control structures (gates, weirs, pump stations) as they exist in the field

In general the network extent is adequately defined by the 10-inch pipes; therefore, only a few 8-inch pipes are included in the model. There are no pump stations or other control structures in the Outfall Sewershed.

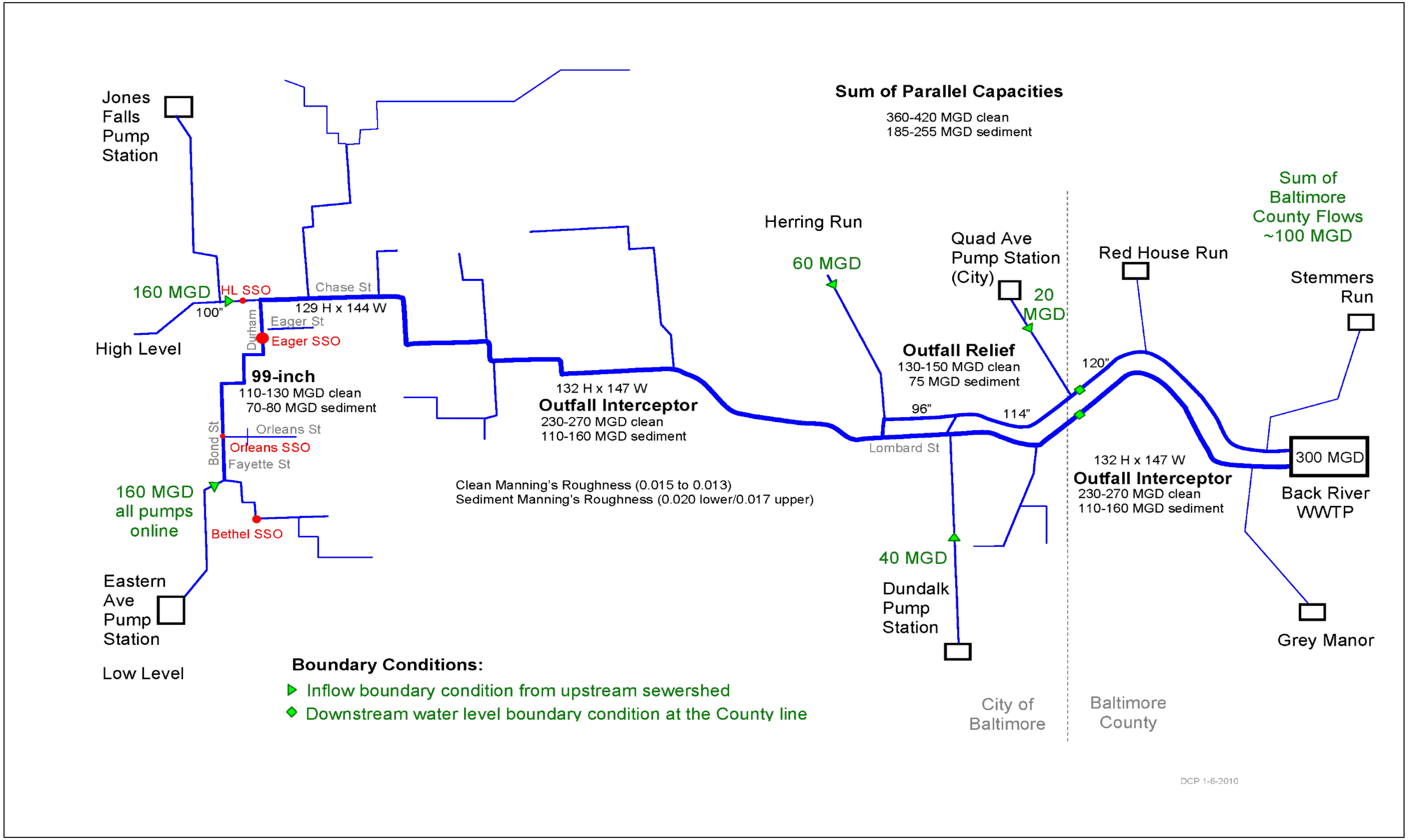


Figure 5.1 Schematic of Outfall Sewershed Trunk Sewers

The configuration of the model is based on GIS data that is developed from field surveyed data (supplemented by as-built drawings). This is to satisfy the Consent Decree paragraph 12.B-(ii)-(a) requirement that the system configuration be based on system attribute data that is representative, accurate and verified.

In accordance with BaSES 7.4.2, the Maryland State Plane Coordinate System (NAD83-Feet) is used for the horizontal datum. The vertical datum for the model is the North American Vertical Datum of 1988 (NAVD88).

Sewer service areas (SSAs) were initially defined by the City, but later refined into model subcatchments to meet the requirements of the CD. The subcatchments are essentially the same as the sewer service areas (SSAs); however, some of the SSAs have been further subdivided to accommodate the need to load flow to each branch of the network. SSAs were also divided due to the locations of the flow meters. These changes follow the guidelines from Section 7.4.4 of the BaSES Manual.

For calibration, the subcatchments are grouped according to the flow meter to which they are tributary. In general, all subcatchments within a flow meter basin have the same calibration parameters.

Sanitary flow and infiltration and inflow (I/I) contribute to the total flow conveyed by the collection system. The model defines the sanitary flow (with a diurnal pattern) and the I/I response to wet weather using parameters in the model subcatchments.

Dry weather flow is discussed in the BaSES Manual Sections 3.2.3, 3.5 and 7.4.5. The objective of the dry weather flow development is to characterize the dry weather flow pattern so that during wet weather conditions it is possible to distinguish between flow due to infiltration and inflow (I/I) and the base sanitary flow. Calibration objectives focus on properly simulating the volume, diurnal peaks, and the timing of the diurnal pattern during dry weather conditions. Water consumption patterns and groundwater infiltration vary over time; much of the variability is periodic and repeated. The model development aims to represent the typical quantity and variability of dry weather flow by a fixed set of parameters. The model cannot duplicate all of the flow patterns or periods of irregular flow; instead it is an approximate match to the dominant dry weather flow characteristics.

Base sanitary flows (BSF) have been developed by the City’s Technical Program Management Team for each SSA (as described in the BaSES manual section 7.4.5). The BSF values represent the sanitary flow generated by users. The average dry day flow (ADF) from each SSA is the sum of the BSF and any groundwater infiltration (GWI).

ADF = BSF + GWI

The ADF is estimated from the flow meter data stored in Sliicer. The GWI is estimated as a calibration parameter to achieve a good match between the simulated flows and the measured flows during dry weather. The estimated GWI determined for each meter basin is assigned to the tributary subcatchments in proportion to area.

BaSES manual section 7.4.6 defines the modeling approach used to simulate the wet weather flow. This modeling approach assumes a direct relationship between rainfall and the wet weather flow response in the sewer system. The details of this deterministic relationship are described below; however, it is important to note that the modeling approach does not account for variable antecedent soil moisture conditions. The model calibration assumes that the hydrologic conditions experienced during the monitoring period are representative of typical hydrologic conditions. Special hydrologic conditions may not be properly modeled using this methodology (such as, events with significant snow melt, back to back events with a prolonged series of significant storms, or extreme events such as hurricane related storms).

The development of the model is based on rainfall and flow meter data. Uncertainties in both the rainfall and flow meter measurements are compounded in the process of developing a model relationship between the two. The uncertainties are not just due to the accuracies of the instruments, but also to the intrinsic variability of the quantities being measured. For example, rainfall measured at a gauge may or may not be a sufficient representation of the rainfall over the meter basin to which it is assigned. The rainfall and flow are measured at spatially separate locations. Overall, the correlation can be derived from the data by calibrating the model to many events. The objective of the calibration is to choose model parameters that realistically characterize the basin response to rainfall for the most probable conditions, even though the match may not be ideal for each and every event in the measurement record. The use of radar rainfall estimates seeks to improve the correlation between rainfall patterns and flow meter response, but rainfall is just one of many sources of variability.

Rainfall derived infiltration and inflow (RDII) is simulated using the SWMM RUNOFF routines in InfoWorks™. The following parameters are needed for each subcatchment in the model to develop wet-weather flows:

- Area
- R-Value
- Depression Storage
- Width
- Slope
- Overland Flow Routing Coefficients

Area

The **Contributing Area** parameter represents the area of each subcatchment, in acres, that is served by the collection system. Areas that are not sewered (i.e. cemeteries, golf courses, parks, etc.) are deducted from the total area of subcatchments to determine the contributing area.

R-Value

The R-Value represents the fraction of the rainfall that enters the sewer system. Sliicer provides an initial estimate of the R-Value for each flow meter basin by plotting the RDII volume versus the rainfall depth (Q vs. I plot) and then developing the best-fit linear regression line (the R-Value is based on the slope of the regression line). In the InfoWorks^{TM} model, the R-Value is input as the **Fixed Runoff Coefficient**. Once in the model, this coefficient may be adjusted to refine the calibration based on the routed simulated response in the model. This provides a more accurate prediction of flow volume.

The equation for I/I volume using the R-value is:

V = K R A (D-DS)
Where V = Volume of I/I
K = a unit conversion constant = 1 MG/36.8 acre inches
R = dimensionless ratio of RDII volume to rainfall volume
A = contributing metershed area (acres)
D = rainfall depth (inches)
DS = depression storage (otherwise known as initial rainfall abstraction) (inches)

Depression Storage

Depression storage represents the amount of rainfall (inches) that is lost to surface wetting, ponding, interception, and evaporation during a storm; this parameter is also commonly known as the "initial abstraction". Depression storage is estimated by the location where the linear regression line intercepts the x-axis of the Sliicer software's Q versus I Plot. Typical values range from 0 to 0.5 inches, but can vary greatly for the same area depending on the antecedent moisture conditions. The depression storage value is entered into the appropriate Runoff Surface under the **Initial Loss Value** field of the InfoWorks^{TM} model.

Width

The subcatchment width, known as the **Dimension** value in InfoWorks^{TM}, is a key calibration parameter that does not have a direct correlation to the actual dimensions of the subcatchment. During calibration, the subcatchment width value is adjusted so that the magnitude and time-to-peak of the simulated flow matches the observed peak flow in the monitoring data (peak RDII flow) for several storm events. Subcatchment width can greatly alter the shape of the hydrograph without impacting the volume. Because the width is directly proportional to the peak flow rate, its value may be adjusted as necessary to match the observed peak flows.

Slope

The **Slope** parameter is given a nominal value similar to the physical slope of the ground surface, but when the SWMM model is being used to simulate RDII, this parameter is no longer physically-based. Slope is a not a sensitive calibration parameter.

Overland Flow Routing Coefficient

The **Overland Flow Routing Coefficient,** also known as the Manning's Roughness Coefficient (n), is a secondary parameter that can be used to alter the shape of the hydrograph. A nominal value of 0.013 was used in the model for all subcatchments; however, this is not a sensitive parameter.

5.2 Model Calibration

BaSES manual Section 7.5 defines the objectives and criteria to be used for the calibration of the dry and wet weather flows. The calibration compares the simulated flows and water levels in the InfoWorks™ model to the measured flows and levels at the monitoring sites. A schematic of the meterbasins, previously given in Section 3, Figure 3.2.1, shows the relationship between the flow meters.

Subcatchments along the branch sewers in the Outfall Sewershed are calibrated using the meters located on the branch sewers. The remaining SSAs tributary to the major trunk sewers used nominal parameters to generate dry and wet weather flows. Meters located on the major trunk sewers are used to calibrate the large scale hydraulic properties and responses of the model (such as roughness, sediment, boundary conditions, and water depth). Thus there are two distinct applications of flow meter data to the model calibration; the smaller branch meters are used to calibrate the SSA flow generation parameters and the larger trunk meters are used to calibrate the large scale hydraulic parameters.

Attachment 5.2.1 is the Model Development and Calibration Report (MDCR) which contains complete details of the model development and the calibration performance.

Dry Weather Calibration

The dry weather calibration criteria are from BaSES manual Section 7.5. For a representative dry weather period, the simulated volume of flow should be within -10% to +20% of the measured volume and the peak dry weather flow rate should be within -10% to +20% of the measured flow rate. The timing of the peaks of the diurnal pattern should be within 1 hour of the measured peaks. Subjectively, the general shape of the diurnal pattern should be representative for most of the dry weather conditions.

The branch sewer meters were used to calibrate the SSAs; for these meters the dry weather comparison of the simulated results to the measured values is given in Table

5.2.1. Five dry weather periods (representing a sum of 114 days of dry weather flow) were used to develop the dry weather calibration parameters. The values presented in Table 5.2.1 summarize the results for a 7-day validation period from 12/4/2006 to 12/10/2006 (except for two meter sites that use other periods as is explained further below). In most cases the results satisfy the calibration criteria given in the BaSES manual; exceptions are explained in the MDCR.

Table 5.2.1 - Dry Weather Calibration Branch Sewer Meters Used to Calibrate Sewer Service Areas							
	Peak Flow				Volume (7 day duration)		
Meter	Measured (MGD)	Simulated (MGD)	Difference (MGD)	Percent Difference	Measured (MG)	Simulated (MG)	Percent Difference
HL01	0.43	0.36	-0.07	-16%	1.86	1.94	5%
HL02	0.67	0.64	-0.03	-4%	3.56	3.78	6%
HL03	0.80	0.89	0.08	10%	4.42	5.24	19%
HL04	0.64	0.60	-0.04	-6%	3.32	3.40	2%
HL05	0.40	0.39	-0.01	-2%	6.78	6.81	0%
OUT01	0.53	0.53	0.00	-3%	5.77	6.04	5%
OUT05	No data	0.19			No data	1.18	
OUT07	0.33	0.38	0.04	13%	1.29	1.82	41%
OUT08	0.62	0.60	-0.02	-4%	3.26	3.36	3%
OUT09	0.55	0.39	-0.16	-28%	1.87	1.82	-2%

Meter HL01 has a unique flow pattern with a strong weekly cycle that does not conform simply to a typical weekday/weekend pattern. Because of this, it is difficult to represent this pattern in the InfoWorks™ model. The selected calibration is a reasonable compromise to adapt the model diurnal flow pattern to the measured flow pattern.

Meters HL02, HL03, HL04 and HL05 are calibrated within the criteria for dry weather flow. Limited data was available for OUT01 beginning in February 2007; the results in the Table 5.1 are based on a dry weather period from 4/25/2007 to 5/10/2007.

No valid flow meter data is available for OUT05; the SSAs tributary to this meter basin have been assigned a flow that is two times the base sanitary flow (BSF) values provided by the City.

Meters OUT07 and OUT09 monitor the same area; OUT09 is a FlowShark meter located a few blocks downstream of OUT07 which is an Isco meter. The peak flow rate and volume at OUT07 and OUT09 should in principle be the same. The flow at these meter sites can be influenced by high water levels in the 99-inch Sewer that is downstream of this branch. The flow and water level in the 99-inch Sewer are largely controlled by the operations of the Eastern Avenue Pump Station. The flow meter data at OUT07 and OUT09 show the influence of the operations of the pump station. In general, it appears that OUT07 yields a better estimate of the peak flow rate while OUT09 yields a more consistent estimate of the volume of flow. The calibration results do not conform to the calibration criteria because of the uncertainty in the measured data.

Table 5.2.2 contains the dry weather calibration results for meters located on the major trunk sewers. The volumes in the table are for the 7-day period 12/4/2006 to 12/10/2006. These meters were used to calibrate the overall hydraulic response of the sewer network. The simulation results at meters along the major trunk sewers are highly sensitive to the assumed boundary condition values. Gaps or irregularities in the measured data used for the boundary condition propagate through the model.

The primary conclusion from this comparison is that the model is properly routing the input flow boundary conditions from the upstream sewersheds (that is, the measured flows from High Level, Jones Falls, Low Level, Herring Run, and Dundalk). The secondary benefit of this comparison is observations about the hydraulic consistency of the measured flow data. Most of the meters used in the large trunk sewers are FlowShark area-velocity meters; three of the meters are Isco area-velocity. In general, the FlowShark meters are better able to monitor the velocity in large pipes than the Isco meters (which are well suited to monitor flow in smaller pipes). Specific observations are noted below (progressing from the upstream to the downstream end).

Table 5.2.2 - Dry Weather Calibration

Major Trunk Sewer Meters Used to Evaluate Overall System Hydraulics (meters ordered from upstream to downstream)

	Peak Flow				**Volume (7 day duration)**		
Meter	**Measured (MGD)**	**Simulated (MGD)**	**Difference (MGD)**	**Percent Difference**	**Measured (MG)**	**Simulated (MG)**	**Percent Difference**
OUT06A[1]	32.41	45.80	13.39	41%	152.13	180.86	19%
OUT06[2]	45.73	44.08	-1.65	-4%	179.82	184.40	3%
TSHL01[2]	92.28	90.85	-1.43	-2%	517.98	521.66	1%
OUT04A[1]	64.87	91.59	26.71	41%	323.49	526.90	63%
OUT04[1]	82.69	91.26	8.57	10%	374.19	528.81	41%
OUT03[2]	90.69	91.41	0.72	1%	516.26	540.68	5%
OUT02[2]	91.40	90.38	-1.02	-1%	519.53	544.68	5%
TSOUT02[2]	100.00	76.40	-23.60	-24%	541.13	465.98	-14%
TSOUT01A[2]	99.20	76.87	-22.33	-23%	582.08	466.22	-20%
TSOUT01B[2]	44.66	40.36	-4.30	-10%	265.30	233.82	-12%

[1]Isco flow meter

[2]FlowShark meter

Meters OUT06A and OUT06 are located on the 99-inch Sewer that conveys flow from the Eastern Avenue Pump Station to the Outfall Sewer. OUT06A is located near the upstream end of the 99-inch Sewer close to the connection of the force main from the pump station. OUT06 is located near the downstream end of the 99-inch Sewer before connecting to the Outfall Sewer. The average dry weather flow in the 99-inch Sewer is approximately 26 MGD, but the flow is highly variable because the flow pattern is dominated by the pump station operations (typically varying from 10 to 40 MGD). The incremental flow from SSAs in the Outfall Sewershed between OUT06A and OUT06 is relatively small (only 0.2 MGD); therefore, the total flow at the two meters is essentially the same.

Meter TSHL01 is a FlowShark meter located near the upstream end of the Outfall Interceptor after the confluence of flows from the High Level/Jones Falls sewersheds and the 99-inch sewer from the Low Level sewershed. The average dry weather flow is approximately 74 MGD. As this data was used to develop the input boundary condition flows, the simulated and measured data agree very closely at TSHL01.

Meters OUT04A and OUT04 are Isco meters located on the Outfall Interceptor. The measured velocities (and consequently the recorded flow rate values) are consistently lower than values at neighboring meters (TSHL01 upstream and OUT03 downstream, both of which are FlowShark meters). It is the opinion of the hydraulic modeling engineers that the flow data at OUT04A and OUT04 have a low bias. It is assumed that the depth data is reasonable and that the flow pattern is realistic, but that measured flow values are lower than actual flows.

Meters OUT03 and OUT02 are FlowShark meters located along the Outfall Interceptor along Monument Street and Lombard Street, respectively. The simulated flows match the measured flows very well at both meter sites. OUT02 is located just upstream of the chamber that allows flow to divide between the 132-inch Outfall Interceptor and the 114-inch Relief Sewer. The average dry weather flow at OUT02 is approximately 78 MGD.

Flow from the Herring Run sewershed enters the model at the upstream end of the Outfall Relief sewer; the average dry weather flow at meter HR01 is approximately 18 MGD. Flow from the Dundalk sewershed enters the Outfall Interceptor just downstream of an inter-connection structure between the Outfall Interceptor and Outfall Relief Sewers. The average dry weather flow at meter TSDU03 is approximately 4 MGD. Based on this information, the sum of the flows from Herring Run, Dundalk and the Outfall Sewershed is approximately 100 MGD; this flow is conveyed by the parallel pipes (Outfall Interceptor and Outfall Relief Sewer) to the Baltimore County Line which is the downstream end of the Outfall Sewershed model.

Meters TSOUT01A and TSOUT01B are FlowShark meters located on the Outfall Interceptor and Outfall Relief Sewer, respectively, near the Baltimore County Line. The balance of flow between the two pipes is very sensitive to the water level boundary condition defined at the Baltimore County Line (the downstream nodes of model). In general the level, velocity, and flow data recorded for TSOUT01A and TSOUT01B are reasonable. The flow at both meter sites is regulated by the water level at the Back River WWTP. The flow at the County Line is subject to backwater conditions from the plant; the depth and velocity relationship does not follow the normal Manning's relationship for open channel flow (the depths are deeper and the velocities are slower than normal flow). This backwater influence is also apparent in the data for all of the major trunk sewer meters.

In addition to the balance of flow between the Outfall Interceptor and Outfall Relief sewer, there is some uncertainty in the magnitude of the measured flows. The measured average dry weather flows are 83 MGD at TSOUT01A and 38 MGD at TSOUT01B; the sum of the flows is 121 MGD. For comparison, the simulated average dry weather flows are 67 MGD at TSOUT01A (19% less than measured) and 33 MGD at TSOUT01B (13% less than measured); the sum of the simulated flows is 100 MGD (17% less than measured). Further efforts to refine the model calibration to increase the simulated flow at the County Line were not pursued because it would make the model less consistent with the other trunk sewer meters along the Outfall Interceptor. It is likely that there is a high bias in the measured flow values at TSOUT01A. This suspicion is supported by the data at meter TSOUT02.

Meter TSOUT02 is FlowShark meter located on the Outfall Interceptor just downstream of the connection from the Dundalk Sewershed. The flow at TSOUT02 and TSOUT01A are, in principle, equal flows. The measured average dry weather flow at TSOUT02 is 77 MGD, which is 6 MGD less than the measured flow at TSOUT01A. This further supports the assumption that meter TSOUT01A has a high bias in the measured flow values.

The simulated flows are consistent with the sum of the measured flows entering the parallel pipes from Herring Run, Dundalk, and the Outfall Sewershed (which is 97 MGD). Therefore, the calibration was defined to agree with as many meter sites as possible; in this case, however, the differences can be seen most clearly in the percent difference between simulated and measured values at TSOUT01A and TSOUT1B. In reality, the uncertainty could be (and likely is) shared between the various meters in the vicinity of the parallel sewers. The model configuration is, in the judgment of the Joint Venture engineers, a realistic representation of the flows and boundary conditions present in the system. The places where the difference between the simulated and measured values exceeds the calibration criteria are acceptable. This discussion of the dry weather flow response also assists with a proper interpretation of the wet weather calibration results.

Wet Weather Calibration

The wet weather calibration seeks to determine parameters that characterize the response in the sewer systems to wet weather conditions that cause I/I. During the 12-month calibration period (May 2006 to May 2007) there were 29 wet weather events identified as global storms. The radar rainfall data (CALAMAR) was used (when available) to drive the model simulations. Ground based rain gauge data was also used to run the simulations; this result provides a check on the radar rainfall simulation.

In general there are two types of significant wet weather events: (1) those that are driven by high intensity rainfall of a relatively short duration and (2) those that are driven by low intensity, longer duration rainfall. Because the modeling system chosen for this effort does not account for the influence of variable soil moisture storage, the simulated flows can either be calibrated to better match the short/high intensity storms or the longer/low intensity storm. The limitations of the modeling approach can not account for a wide variety of hydrologic conditions. Preference is given in this calibration to short/high intensity storms which drive the highest peak flows. This is also the type of event that was used to evaluate system capacity, as described later in this section.

The wet weather calibration criteria from the BaSES manual Section 7.5.2 are summarized in Table 5.2.3. In addition to the flow related comparisons (peaks and volumes), there are also criteria to evaluate the depth of flow. For pipes that are not surcharged, the simulated depth of flow should be within 4 inches of the measured depth of flow. For surcharged pipes the criteria depends on the size of pipe and whether the simulated flows are greater than or less than the measured depths.

Table 5.2.3: Wet Weather Validation Criteria

Simulated response	Percent difference from observed measurements
Peak Flow Rate	Within –10% and + 25%
Volume of Flow (assume duration from the start of rainfall to 2 days after rainfall ends)	Within –10% and + 20%
Depth of Flow in Surcharged Pipes: For pipes 21-inch diameter and larger For pipes smaller than 21-inch diameter	 Within -4 inches and +18 inches Within -4 inches and +6 inches
Depth of Flow in Unsurcharged Pipes	Within 4 inches
Shape and timing of hydrographs	Should be similar

The calibration results for each flow meter location are summarized in the MDCR with a time series plot and three statistical plots that compare the simulated results to the measured values. The statistical plots are a concise summary of the results that show the correlation between simulated results and observed values. Using meter OUT08 as an example, Figures 5.2.1, 5.2.2, and 5.2.3 are statistical plots for peak depth, peak flow, and volume for the wet weather events. Each statistical plot has a one-to-one line that represents perfect correlation between simulated and observed values. Upper and lower reference lines on the statistical plots show the envelope of the calibration criteria. When the pipe is not surcharged, the calibration criterion for peak depth is ±4 inches. When the pipe is surcharged, the calibration criterion is +18 inches and -4 inches because the pipe size is 24-inches. If the pipe diameter were less than 21 inches, the surcharged criteria would be +6 inches and -4 inches. Reference lines also mark the pipe crown to show surcharging when peak depth are greater than the pipe diameter.

For the larger surcharged events, the simulation results are within the calibration boundaries. When the pipe is not surcharged, the simulated peak depths are generally greater than observed depths. For a few of the smaller events in the transition zone, the model tends to simulate surcharging conditions for some events that did not have observed surcharging.

Each statistical plot shows the data points and two regression lines that have been fitted to the data points. One of the regression lines assumes a y-intercept of zero and the other allows for a y-intercept offset value. The equation and the goodness of fit correlation coefficient, R^2, are printed on the graph for each regression line. The correlation coefficient, R^2, is an indication of how well the model fits for a variety of wet weather conditions.

Figure 5.2.1 shows the simulated peak hydrograph depth compared to the measured flow depth. The simulated depths are typically higher during low flow periods, but are within the calibration criteria for the larger events with surcharging.

In Figure 5.2.2 the slope of the dotted red line for peak flow is 0.99, which means that the simulated peak flows are very close to the observed values overall. Reference lines on the plots of peak flow mark the calibration criteria of +25% and –10%.

In Figure 5.2.3 the slope of the dotted red line for the event volume is 1.05, which means that the simulations over predict the event volume by 5% on average. Reference lines on the statistical plots of event volume mark the calibration criteria of +20% and -10%.

Overall, the calibration of SSAs in meter basin OUT08 produce simulated results that are a realistic representation of the actual system hydraulics. The calibration results at OUT08 are representative of the overall calibration of the model at the other meter sites as well.

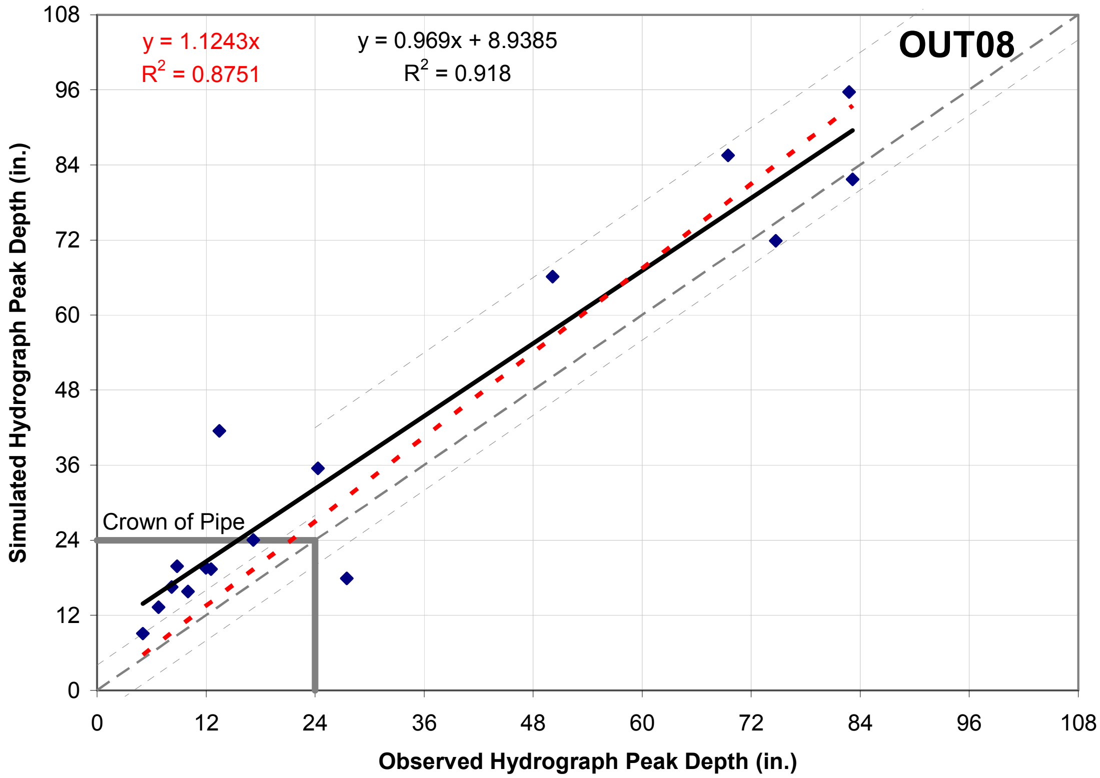


Figure 5.2.1. Statistical Plot of Peak Depth for OUT08.

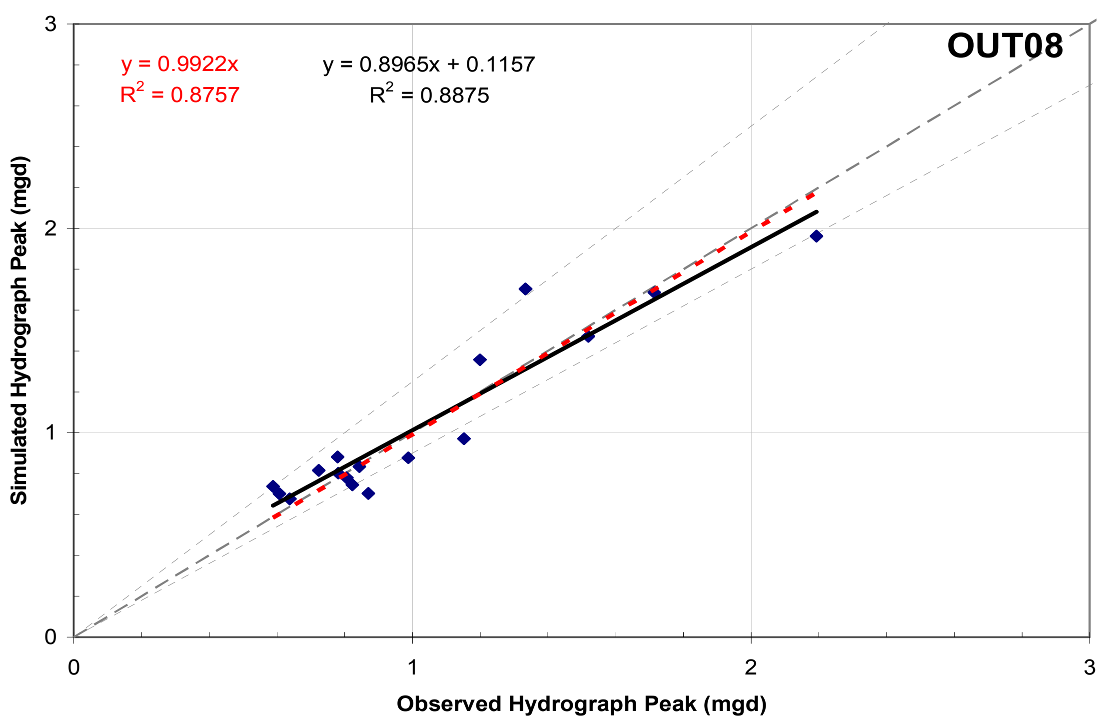


Figure 5.2.2. Statistical Plot of Peak Flow for OUT08.

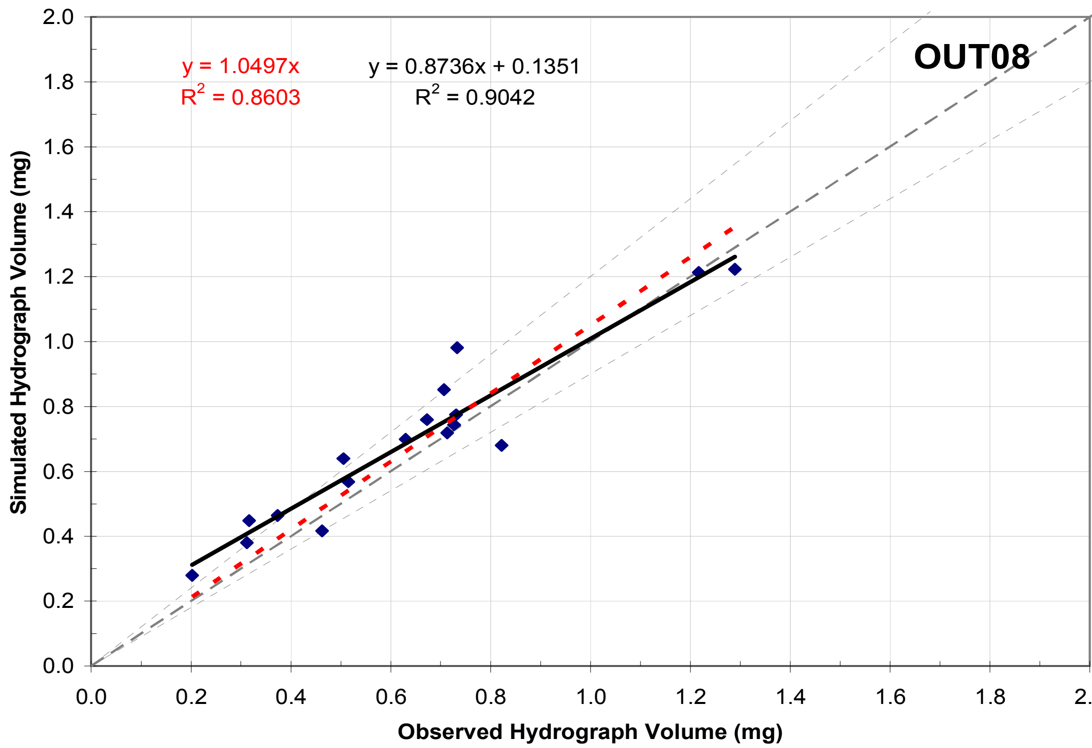


Figure 5.2.3. Statistical Plot of Event Volume for OUT08.

Table 5.2.4 is a very brief summary of the wet weather calibration results for all meter sites. For most of the meters, the average trends of simulated values are within the calibration criteria. For meters OUT04, OUT04A, and OUT06A, the measured values are unrealistically low (compared to neighboring meters) and this is the reason that the simulated values are not within the calibration criteria. For TSOUT01A and TSOUT02 the measured values may be high for peak flow and volume. The simulated values at these sites are very sensitive to the assumed water level boundary conditions used at the Baltimore County line (as discussed above in the dry weather calibration discussion).

For meter HL03, the simulated peak flow values are much higher than the measured peak flows at HL03, but they are consistent with peak measured peak flows upstream at meter HL04. High water levels in the Outfall Sewer cause surcharging at meter HL03, likely were causing the measured peak flow values to be unrealistically low.

Table 5.2.4 - Summary of Wet Weather Calibration

Metershed	Peak Depth	Peak Flow	Volume
HL01	OK	OK	OK
HL02	OK	OK	OK
HL02B	OK	OK	OK
HL03	OK	Measured Values Low	OK
HL04	OK	OK	OK

Table 5.2.4 - Summary of Wet Weather Calibration

Metershed	Peak Depth	Peak Flow	Volume
HL05	OK	OK	OK
OUT01	OK	OK	OK
OUT02	OK	OK	OK
OUT03	OK	OK	OK
OUT04	OK	Measured Values Low	Measured Values Low
OUT04A	OK	Measured Values Low	Measured Values Low
TSHL01	OK	OK	OK
OUT05	N/A	N/A	N/A
OUT06	OK	OK	OK
OUT06A	OK	Measured Values Low	Measured Values Low
OUT07	OK	OK	OK
OUT08	OK	OK	OK
OUT09	OK	OK	OK
TSOUT02	Measured Values Low	Measured Values High	Measured Values High
TSOUT01A	OK	Measured Values High	Measured Values High
TSOUT01B	OK	OK	OK

5.3 Baseline Analysis and Capacity Assessment

The Baseline Analysis and Capacity Assessment (BACA) is an evaluation of the hydraulic performance of the sewer network in the Outfall Sewershed for baseline and future 2025 conditions during dry weather and wet weather conditions. Guidelines and requirements for the BACA are provided in the Baltimore Sewer Evaluation Standards (BaSES) Manual Sections 7.6.3 and 7.8.2. The analysis is based on simulated hydraulic model results that assess the capacity of the system for baseline and future conditions for a variety of design storms.

Baseline conditions are comprised of the existing sewer infrastructure and flow based on the 2007 population and land use conditions. Future 2025 conditions in the Outfall Sewershed have the same sewer network as the Baseline conditions except for the disconnection of the 15-inch pipe serving meter basin OUT05. The City performed a project in 2009 to connect this 15-inch pipe to the Low Level Sewershed. Subcatchment flows from the Outfall Sewershed are based on estimates of the future 2025 population and land use; this produces an 8.5% increase from the baseline base sanitary flow rates. The degradation of the sewer system is modeled as a 10% increase in the groundwater infiltration rate. Other features of the Baseline model remain the same in the Future 2025 model, such as the wet weather flow characteristics of the subcatchments and the sediment levels.

Subsequent to the writing of the BACA report, the Future 2025 boundary conditions were revised to account for proposed conveyance system improvements in the upstream sewersheds. This set of boundary conditions are designated “Upstream Improvements” and the results of the future capacity analysis using these boundary conditions is reported in the AARR and summarized in this Sewershed Plan.

Attachment 5.3.1 is the BACA report and its associated appendix documents.

5.3.1. Design Storms

Rainfall depths related to specific design storms are published by the National Weather Service (1). The time varying rainfall patterns (hyetographs) of the design storms were defined by the technical program manager based on the NRCS/NOAA rainfall distribution (2). Table 5.3.1 lists the design storms to be used for the BACA. These events are defined in BaSES Manual Section 7.6.1 and fulfill the requirements of CD paragraph 9 F ii.

Table 5.3.1 - Rainfall Design Storms

Rainfall Recurrence Interval	Rainfall Duration	Rainfall Depth (inches)
3-month	1 hour*	1.11
1-year	24 hour	2.67
2-year	24 hour	3.23
5-year	24 hour	4.15
10-year	24 hour	4.97
15-year	24 hour	5.41
20-year	24 hour	5.82

* Approximately equal to the time of concentration of the Outfall Sewershed subcatchments.

Figure 5.3.1.1 shows the rainfall hyetographs in which the peak rainfall intensity starts just after noon in the middle of the 24-hour duration.

This distribution is intended to represent an event that has the same rainfall recurrence interval for several duration periods. For example, Figure 5.3.1.2 shows the depth-duration-frequency relationship for the 5-year recurrence interval event. The rainfall depth satisfies the 5-year recurrence interval not only for the overall 24-hour duration, but also for 1, 3, 6, and 12 hour durations. The benefit of this type of rainfall distribution is that subcatchments of various sizes (and various times of concentration values) experience a rainfall input that has an equal frequency of recurrence.

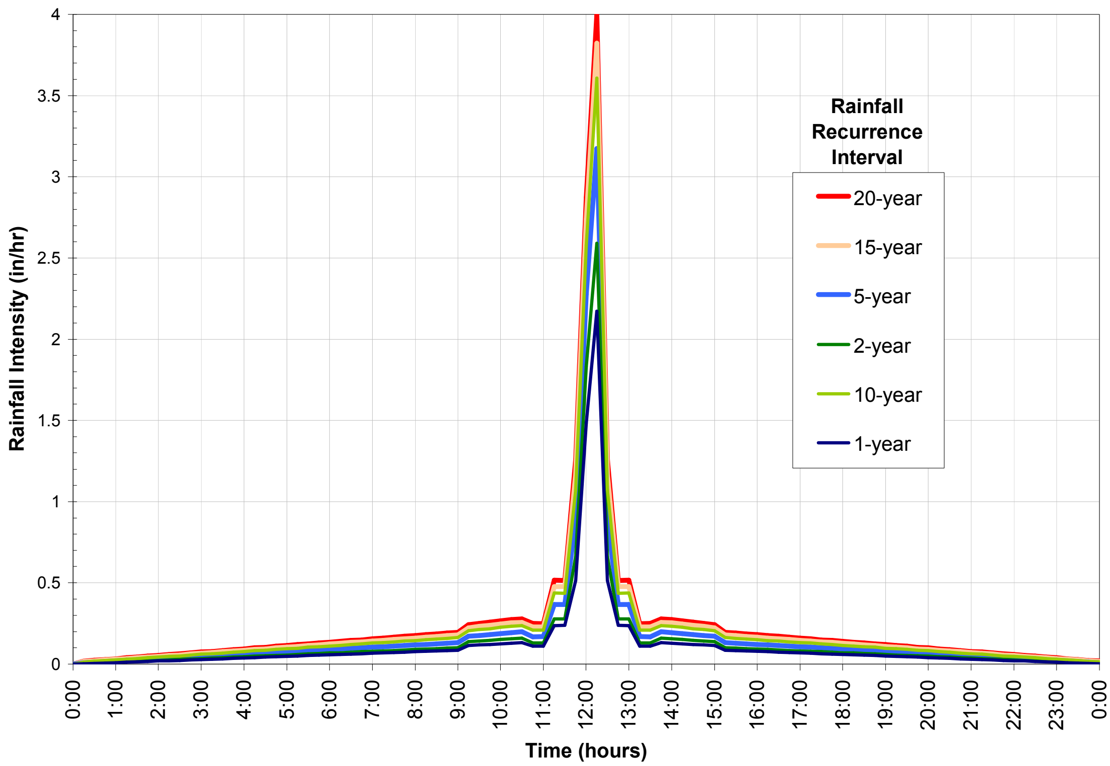


Figure 5.3.1.1: Design Storm Rainfall Hyetographs

5.3.2. Definition of Deficiency

Capacity is defined in the BaSES manual, Section 7.6, as the level of service which the system can provide without an overflow. Surcharging is allowed as long as water levels do not exceed manhole rim elevations.

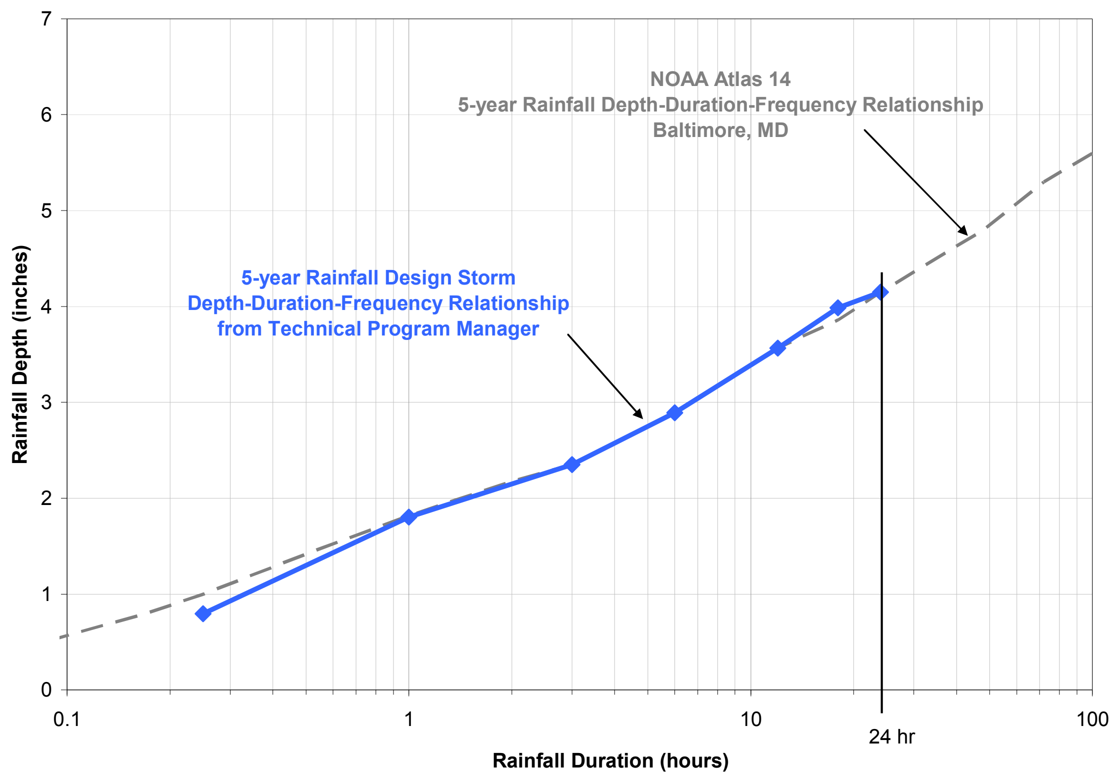


Figure 5.3.1.2: Design Storm Rainfall Depth-Duration-Frequency Relationship

5.3.3. Storm Simulations (All Storms)

This section summarizes the risk of overflows in the Outfall Sewershed by identifying SSO locations and quantifying the SSO risk in terms of simulated SSO volume for the various design storms. The next section, 5.3.4-Identification of Hydraulic Deficiencies, identifies the pipe segments that do not have adequate conveyance capacity. These pipes contribute to the cause of the SSOs.

The BACA evaluation of the Outfall Sewershed uses two alternative scenarios:

- The Active Boundary Conditions scenario incorporates upstream sewersheds flows into the Outfall Sewershed model and downstream water levels at the Baltimore County line.
- The Inactive Boundary Conditions scenario assumes no input flow from the upstream sewersheds and a free flowing outlet at the downstream boundaries of the Outfall Sewershed model. In this scenario the branch sewers have a free discharge into the trunk sewers, which is necessary to identify overflows in the branch sewers that are caused by hydraulic restrictions in the branch sewers themselves.

The results of the active boundary conditions simulations are strongly influenced by the downstream water level boundary conditions at the Baltimore County line, which in turn depend on constraints at the Back River Wastewater Treatment Plant (WWTP).

Maps identifying the locations of overflows for all seven design storm simulations are available in the appendix of the BACA report (Attachment 5.3.1). The maps indentify overflow locations for Baseline (Year 2007) and Future Year 2025 conditions using Active and Inactive boundary conditions.

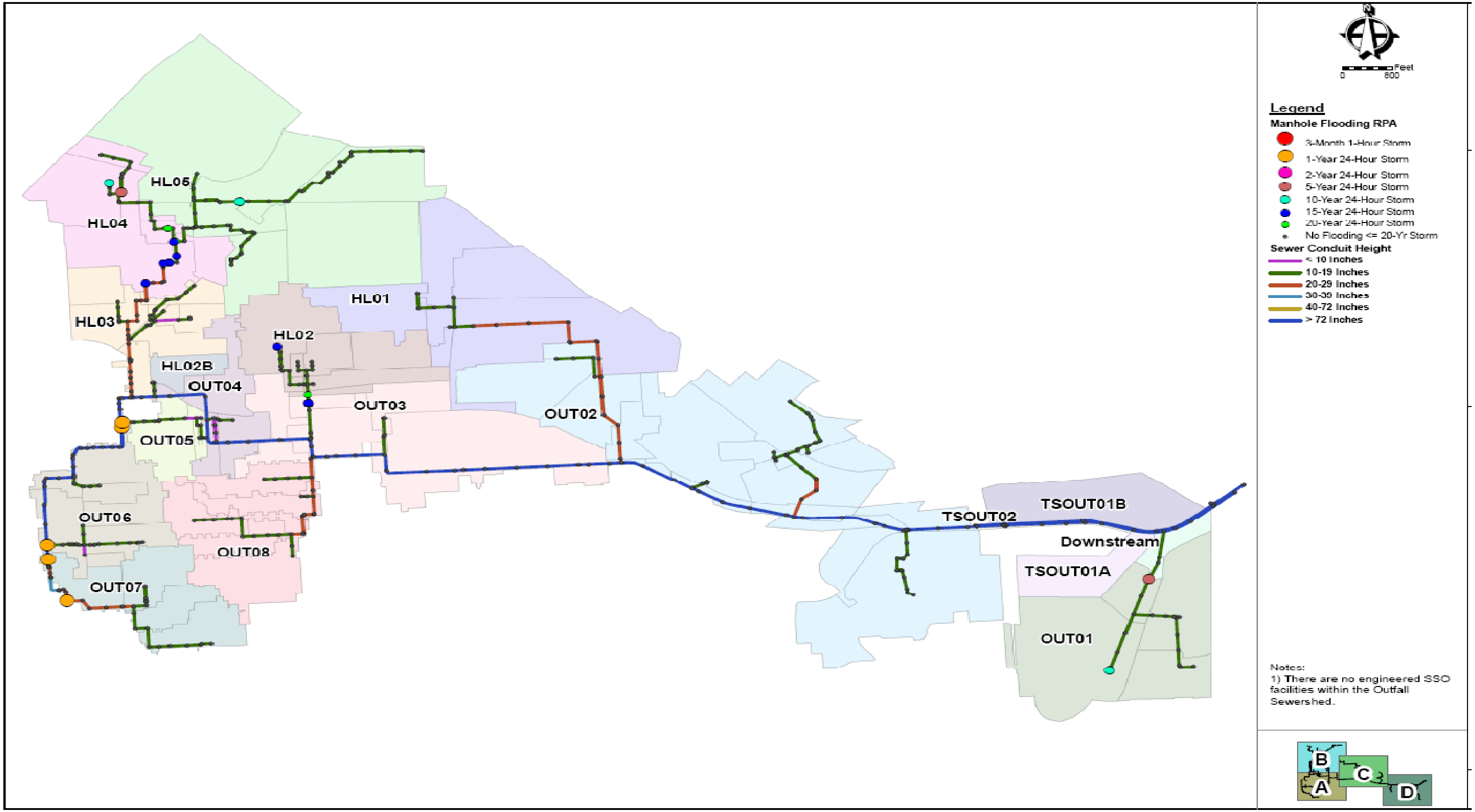
Map 5.3.3 is one of the maps from Attachment 5.3.1 showing locations with simulated SSOs and locations with a risk of SSOs; this particular map is for Baseline Active Boundary Conditions.

Three manholes have the largest SSO volumes among those in Outfall Sewershed. The SSO volumes at the other manholes are relatively small compared to the largest three. Table 5.3.2 lists the SSO volumes at each manhole. The order of the list is ranked from the largest to the smallest volumes. The table also lists the manholes with simulated maximum HGLs within 4 inches of the ground surface; there is no simulated SSO volume at these manholes, they are simply marked as locations where there is a risk of an SSO.

The largest SSO volume is at manhole S45CC_007MH (Durham Street, south of Eager Street) on the 99-inch sewer. This manhole is near the downstream end of the 99-inch sewer, just upstream of the 15-inch connection from OUT05. The second largest SSO volume is simulated at manhole S45CC_021MH (Eager Street, at Durham Street), on the 15-inch sewer serving OUT05, adjacent to the connection with the 99-inch sewer. These two manholes account for approximately 90 to 95% of the total SSO volume in the Outfall Sewershed. (In Future 2025 conditions, the 15-inch Eager Street sewer is disconnected from the 99-inch sewer and is tributary to the Low Level sewershed.)

The third largest SSO volume is at S43E__016MH (Bethel Street and Moyer Street) along the 24-inch branch sewer that serves OUT07. This location is subject to back flow conditions from high water levels in the 99-inch trunk sewer that are strongly influenced by the operations of the Eastern Avenue Pumping Station. The overflows at this manhole account for 5 to 7% of the total SSO volume in the Outfall Sewershed.

Closely associated with the SSO at S43E__016MH is a much smaller SSO volume from the 99-inch sewer at manhole S43A__038MH (Bond Street at Orleans Street). The volume of overflow at Orleans Street is roughly 2% of the volume at Bethel Street. Both overflow locations provide relief to the system near the upstream end of the 99-inch sewer and are driven by high pumping rates from the Eastern Avenue Pump Station.



Map 5.3.3 Simulated Overflow Locations for Baseline Conditions

At the time of writing the BACA report, the boundary conditions to be applied to the Outfall Sewershed model for the Future 2025 conditions did not reflect improvements to facilities in upstream sewersheds that had the potential to increase flows to the Outfall Sewershed. Subsequent to that time, the Future 2025 boundary conditions were refined by the Technical Program Manager to reflect the recommended upstream improvements. The Future 2025 results in the original BACA report are useful in that they identify locations with a SSO risk and sections of pipes that have hydraulic restrictions. The qualitative results are informative, but the numerical magnitude of overflow volumes and peak overflow rates in the original BACA report for the Future 2025 condition are based on the original boundary conditions which produce significantly smaller simulated overflows.

Simulation results with revised boundary conditions were presented in the Alternatives Analysis and Recommendations Report (AARR). The revised results are referred to as simulations with the "Upstream Improvements" boundary conditions. As a result of these planned upstream improvements, flow hydrographs from the Low Level and High Level sewersheds are significantly larger (in volume and peak flow rate) and the downstream level boundary conditions at the County Line are significantly higher. The simulated overflow volumes are listed in Table 5.3.3. The overflow volumes with Upstream Improvements are the basis for evaluating the performance of alternatives in the next section.

Table 5.3.2 - SSO Volume – Baseline Flooding Return Period Analysis – Active Boundary Conditions								
	Manhole SSO Volume (MG)							
	Rainfall Return Period							
Manhole	**DWF**	**3 mo**	**1 yr**	**2 yr**	**5 yr**	**10 yr**	**15 yr**	**20 yr**
S45CC_007MH			0.952	3.262	6.830	9.734	12.027	13.970
S45CC_021MH			1.812	2.865	3.934	4.651	5.002	5.278
S43E__016MH			0.140	0.396	0.903	1.284	1.327	1.593
S43A__038MH[1]			0.0002	0.0002	0.028	0.070	0.021	0.037
S43C__022MH			risk	risk	risk	risk	risk	risk
S69C__002MH					0.001	0.054	0.088	0.118
S45OO_014MH					0.010	0.037	0.051	0.061
S69G__005MH						0.023	0.051	0.081
S47MM_042MH						0.017	0.041	0.066
S43OO_002MH						0.001	0.006	0.012
S45KK_020MH							risk	risk
S45KK_031MH							0.006	0.016
S49EE_004MH							0.002	0.011
S45KK_026MH							0.001	0.004
S45KK_003MH							0.001	0.004
S49GG_039MH							0.0002	0.007
S45MM_014MH							risk	0.001
S49EE_007MH							risk	risk
S49EE_029MH								0.001
S45MM_002MH								risk
S45MM_018MH								risk
Total SSO			**2.9**	**6.5**	**11.7**	**15.9**	**18.6**	**21.3**

[1]Overflow volume at manhole S43A__038MH is associated with overflow at S43E_016MH.
[2] “Risk” means the simulated water level is within 4 inches of the manhole rim.

Table 5.3.3 - SSO Volume – Future 2025 Flooding Return Period Analysis – Upstream Improvements Conditions							
Manhole	**2-yr**	**5-yr**	**10-yr**	**15-yr**	**20-yr**	**Meter Basin**	**Location**
S45CC_007MH	23.137	33.544	40.447	44.689	46.721	OUT06	Durham Street, south of Eager Street
S45CC_021MH	-	-	-	-	-	OUT05	Eager Street, at Durham Street (Future: Disconnected from Outfall)
S43E__016MH	1.487	2.115	2.595	2.914	3.125	OUT07	Bethel Street and Moyer Street
S43A__038MH	1.275	2.742	3.851	4.481	4.926	OUT06	Bond Street, at Orleans Street
S43C__022MH	0.206	0.741	1.411	1.683	1.983	OUT06	Bond Street, between Orleans Street and Fayette Street
S69C__002MH	0.000	0.003	0.095	0.145	0.189	OUT01	Sewer along RR tracks parallel to and between Kane St and Interstate 95. Behind the City of Baltimore Solid Waste Station at 111 Kane St.
S45OO_014MH	0.000	0.010	0.037	0.050	0.061	HL04	Wolfe Street at Darley Avenue
S69G__005MH	0.000	0.000	0.025	0.053	0.084	OUT01	Railroad tracks between Kane St and Interstate 95, at Eastern Ave.
S47MM_042MH	0.000	0.000	0.017	0.040	0.065	HL05	Sinclair Lane at Homestead Street
S43OO_002MH	0.000	0.000	0.001	0.006	0.012	HL04	Cliftview Avenue, half a block east of Wolfe Street
S45EE_015MH	-	-	-	-	-	near OUT06	Durham Street, south of Chase Street
S45KK_020MH	0.000	0.000	0.000	0.000	0.000	HL04	Lanvale Street, where the sewer turns south along Washington Street
S45KK_031MH	0.000	0.000	0.000	0.007	0.017	HL04	Lafayette Avenue, where the sewer turns south along Castle Street
S49EE_004MH	0.000	0.000	0.000	0.004	0.015	HL02	Luzerne Avenue, at Beryl Avenue
S45KK_026MH	0.000	0.000	0.000	0.002	0.005	HL04	Lafayette Avenue, between Chester Street and Castle Street
S45KK_003MH	0.000	0.000	0.000	0.001	0.005	HL04	Chester Street (west side of street), north of Lafayette Avenue
S49GG_039MH	0.000	0.000	0.000	0.001	0.008	HL02	Milton Avenue, north of Preston Street
S45MM_014MH	0.000	0.000	0.000	0.000	0.001	HL04	Chester Street (east side of street), south of North Avenue
S49EE_007MH	-	-	-	-	-	HL02	Luzerne Avenue, at Beryl Avenue
S49EE_029MH	0.000	0.000	0.000	0.000	0.001	HL02	Luzerne Avenue, between Beryl Avenue and Chase Street
S45MM_002MH	-	-	-	-	-	HL04	Alley parallel to North Avenue and E. 20th Street, between Castle Street and Chester Street
S45MM_018MH	-	-	-	-	-	HL04	Chester Street (west side of street), south of North Avenue
S49GG_032MH	-	-	-	-	-	HL02	Biddle Street, just east of Luzern Avenue
S43C__017MH	0.000	0.000	0.004	0.000	0.014	OUT07	just south of Fayette and Bond
S43C__026MH	0.000	0.000	0.003	0.000	0.011	OUT07	just south of Fayette and Bond
Sum of SSO (MG)	**26.1**	**39.2**	**48.5**	**54.1**	**57.2**		**Total for the Outfall Sewershed only**
S43EE_034MH	3.2	6.2	8.8	9.7	11.2	HL end	High Level Sewershed, Chase near Rutland, just upstream of the Outfall Interceptor
Sum of SSO (MG)	**29.3**	**45.4**	**57.3**	**63.8**	**68.5**		**Total including overflow in High Level at S43EE_034MH**

5.3.4. Identification of Hydraulic Deficiencies (All Storms)

Hydraulic deficiencies are sections of pipe that do not have adequate conveyance capacity; these are also called hydraulic restrictions in the BACA report.

Sediment accumulations in the large trunk sewers (99-inch pipe, Outfall Interceptor, and Outfall Relief sewer) reduce the conveyance capacity. The capacities of the large trunk sewers are not sufficient to convey the peak flows. Even without sediment, the capacities of the large trunk sewers are not sufficient to convey the peak flows used in the simulations.

In certain critical locations, the sewer system within the Outfall Sewershed has little tolerance for surcharging. Several manholes in the vicinity of the junction at the upstream end of the Outfall Interceptor (Chase and Durham Streets) have low ground surface elevations. Manhole S45CC_021 MH (Eager Street, at Durham Street) on the 15-inch pipe from OUT05 has the lowest ground surface elevation in this area. Only 1.8 feet of surcharge at the upstream end of the Outfall Interceptor is possible before manhole S45CC_021MH starts to overflow. Other manholes on the 99-inch sewer and the 100-inch sewer from the High Level Sewershed are also shallow and are at risk of SSOs.

The 15-inch sewer from OUT05 was disconnected from the 99-inch sewer in 2009 and this change is reflected in the Future 2025 model setup. The disconnection eliminates the SSO at manhole S45CC_021MH, but increases the volume of SSO at a nearby manhole on the 99-inch sewer (manhole S45CC_007MH, Durham Street, just south of Eager Street). The volume of flow in the 15-inch pipe is relatively small. The model accounts for the fact the tributary area to the 15-inch pipe no longer contributes flow to the 99-inch pipe, but the impact of this disconnection on the overall hydraulic performance of the Outfall Sewershed is negligible.

Meter basin OUT07 is served by a branch sewer that connects to the upstream end of the 99-inch sewer. Manhole S43E__016MH (Bethel and Moyer Streets) on the 24-inch branch sewer in OUT07 is vulnerable to overflows when the Eastern Avenue Pumping Station is pumping with more than three pumps online. The simulation results show flow reversing in the OUT07 branch sewer when high water levels in the 99-inch sewer are partially relieved by overflowing at manhole S43E__016MH (Bethel and Moyer Streets). Flow meter OUT09 monitors the same branch sewer as meter OUT07. The flow reversal behavior is observable in the raw 5-minute data for OUT09 in the large wet weather event of 11-16-2006.

The branch sewer from OUT01 is vulnerable to overflows at manhole S69C__002MH in the 5-year event because of relatively high flows and a low ground elevation. Manhole S69C__002MH is on the 18-inch sewer along the railroad tracks between Kane Street and Interstate highway 95 (behind the City of Baltimore Solid Waste Station at 111 Kane Street).

Near the upstream end of the HL04 meter basin, manhole S45OO_014MH (Wolfe Street at Darley Avenue) has a simulated SSO in the 5-year event. The low ground elevation over the pipe (cover less than 4 feet) makes this manhole vulnerable to overflows. The overflow is caused by a hydraulic restriction in the 10-inch pipe along Wolfe Street between Darley Avenue and Sinclair Lane.

Manhole S47MM_042MH (Sinclair Lane at Homestead Street) is the location of a SSO for the 10-year event in the HL05 meter basin. The ground surface elevation is approximately 6 feet lower than other manholes along Sinclair Lane. The hydraulic restriction in the 12-inch Collington Avenue line contributes significantly to the cause of this SSO.

The remaining SSO locations are associated with infrequent return period events and high flows all along the length of the branch sewers rather than localized hydraulic restrictions. High water levels in the Outfall sewer in the active boundary condition scenario contribute to the occurrence and severity of these overflows, but the overflow volumes are relatively small.

5.4 Alternative Analysis (2-Year and Larger Storms)

The Alternative Analysis and Recommendation Report (AARR) is a discussion of the development and evaluation of facilities for three alternatives that eliminate SSOs in the Outfall Sewershed. The objectives of the Consent Decree relevant to the AARR are defined in the BaSES Manual, particularly sections 7.7, 7.8.3, and 8.2. The alternatives mitigate SSOs for design storms of increasing severity. Attachment 5.4.1 contains the AARR.

The Outfall Sewershed is unique among all of the Baltimore sewersheds in that most of the flows conveyed through the Outfall Sewershed network originate from upstream sewersheds (Jones Falls, High Level, Low Level, Herring Run, and Dundalk). A relatively small fraction of the flow originates from the subcatchment areas within the Outfall Sewershed. Consequently, the largest and most costly alternative facilities are sized to accommodate the high flows from upstream sewersheds. Conveyance improvements in the upstream sewersheds have the potential to increase the risk of SSOs in the Outfall Sewershed and have a direct influence on the size and cost of the required alternative facilities.

All of the alternatives assume that sediment is removed from the 99-inch sewer, Outfall Interceptor, and Outfall Relief sewer. Sediment removal increases the conveyance capacity by restoring the full cross section area and reducing the hydraulic roughness of pipes.

Alternative 1 proposes two storage tanks, one at Fayette and Bond Streets and the other at Chase and Durham Streets, to attenuate the upstream peak flows. Excess flows enter the storage tanks so that the remaining flows are within the conveyance capacities of the pipes. Alternative 1 does not assume any changes downstream at the Back River WWTP.

Alternative 2 assumes that downstream improvements are in place. These improvements must increase the capacity of the Back River WWTP to receive more flow (by either additional treatment capacity or storage at the plant). Downstream improvements greatly increase the conveyance capacity of the Outfall Interceptor and reduce the volume of storage required at upstream locations in the Outfall Sewershed. As a result no storage is needed for the 2-year event and only one storage tank is needed for the 5, 10, 15, and 20-year events. The tank is located at the Fayette relief site and is much smaller than the size of the tanks used in Alternative 1 for the various storms.

Alternative 3 also assumes that downstream improvements are in place. Alternative 3 uses a tunnel from the proposed Fayette Street relief point to a proposed reconnection point along Lombard Street near to the connection from the Dundalk Sewershed.

For the purpose of this study, the downstream improvements are represented in the Outfall Sewershed model as a downstream level boundary condition at the County Line that does not exceed 48 feet (above NAVD88 datum). At 48 feet the Outfall Interceptor and Outfall Relief sewer are approximately 90% full with the water levels one foot below the crowns of the pipes.

While cleaning the sediment from the pipes helps to restore needed conveyance capacity, the peak upstream flows are anticipated to exceed the conveyance capacity of the clean pipes. The limiting hydraulic feature is the Outfall Interceptor from its upstream end to where the Outfall Relief Sewer Starts. Once the Outfall Relief Sewer runs parallel to the Outfall Interceptor there is sufficient capacity to convey simulated wet weather flows. The existing conditions at the BRWWTP are an additional limitation on the peak flow that can be conveyed by the Outfall Interceptor. Therefore, sediment cleaning is not a stand alone solution to the cause of overflows in the Outfall Sewershed.

The AARR contains results of a sensitivity study that examines the risk of failing to achieve the desired level of protection against overflows due to variations in key modeling parameters. In particular, the study evaluated the sensitivity to hydraulic roughness (Manning's n value) of the pipes after they are cleaned of sediment and sensitivity to the operations of the Eastern Avenue Pump Station (EAPS) during wet weather events. When sediment is removed, the Manning's roughness coefficient (n) is assumed to be 0.013. However, because the results are very sensitive to this assumption, the system performance was also evaluated for a Manning's roughness value of 0.015 to determine the necessary facilities to perform adequately for sub-optimum conditions. The results of the sensitivity analysis are in the AARR.

5.4.1 Description of Trunk Sewer Alternatives

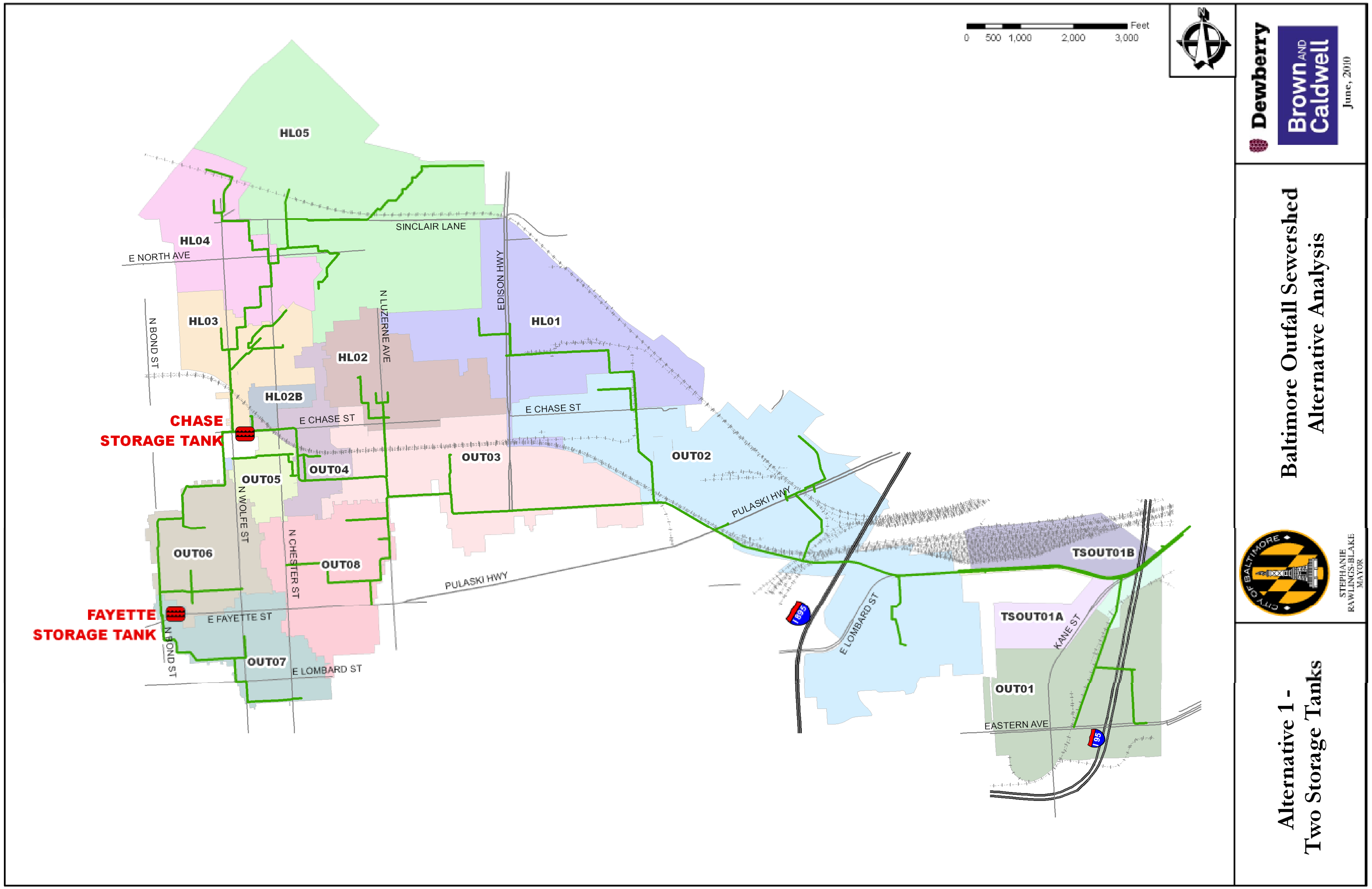
It is assumed that sediment is removed from the trunk sewer in all of the alternatives presented below.

Alternative 1: Storage Using Two Tanks

Alternative 1 uses two storage tanks to store excess flow and prevent SSOs as shown in Map 5.4.1.1. An overflow weir at the upstream end of the 99-inch sewer is needed in the vicinity of Bond and Fayette Streets. This relief facility is called the Fayette weir in the discussion below. The facility should be located between Fayette and Orleans Streets, in close proximity to the connection from the Eastern Avenue Pump Station force main. The purpose of the Fayette weir is to limit the maximum water level at the upstream end of the 99-inch sewer to approximately 58 feet; at this level the 99-inch sewer is surcharged 3 feet and the risk of a SSO further upstream along the 24-inch branch sewer at Bethel and Moyer Streets (manhole S43E__016MH) is minimized.

Relief is also needed to protect the upstream end of the Outfall Interceptor from excessive surcharging in the vicinity of Chase and Durham Streets. The purpose of the Chase weir is to limit the maximum water level at the upstream end of the Outfall Interceptor to no more than 57 feet; at this level the Outfall Interceptor is surcharged 3 feet and the risk of an SSO is reduced at Durham and Eager Streets (manhole S45CC_007MH). The Chase weir should be relatively long to allow significant overflow rates (into a storage tank) with a relatively small head on the weir.

The two storage tanks attenuate the peaks of the inflow hydrographs so that peak flows are within the capacities of the large diameter trunk sewers assuming that the sediment has been removed. Alternative 1 assumes that there are no changes downstream at the Back River WWTP; consequently, the Outfall Interceptor is surcharged to within 2.5 feet of the ground surface at the County Line in the 2-year event. Without improvements at the Back River WWTP, the tanks in this alternative are sized to store the excess flow that can not be conveyed and treated immediately during the event.



Map 5.4.1.1 Alternative 1 Facilities: Two Storage Tanks

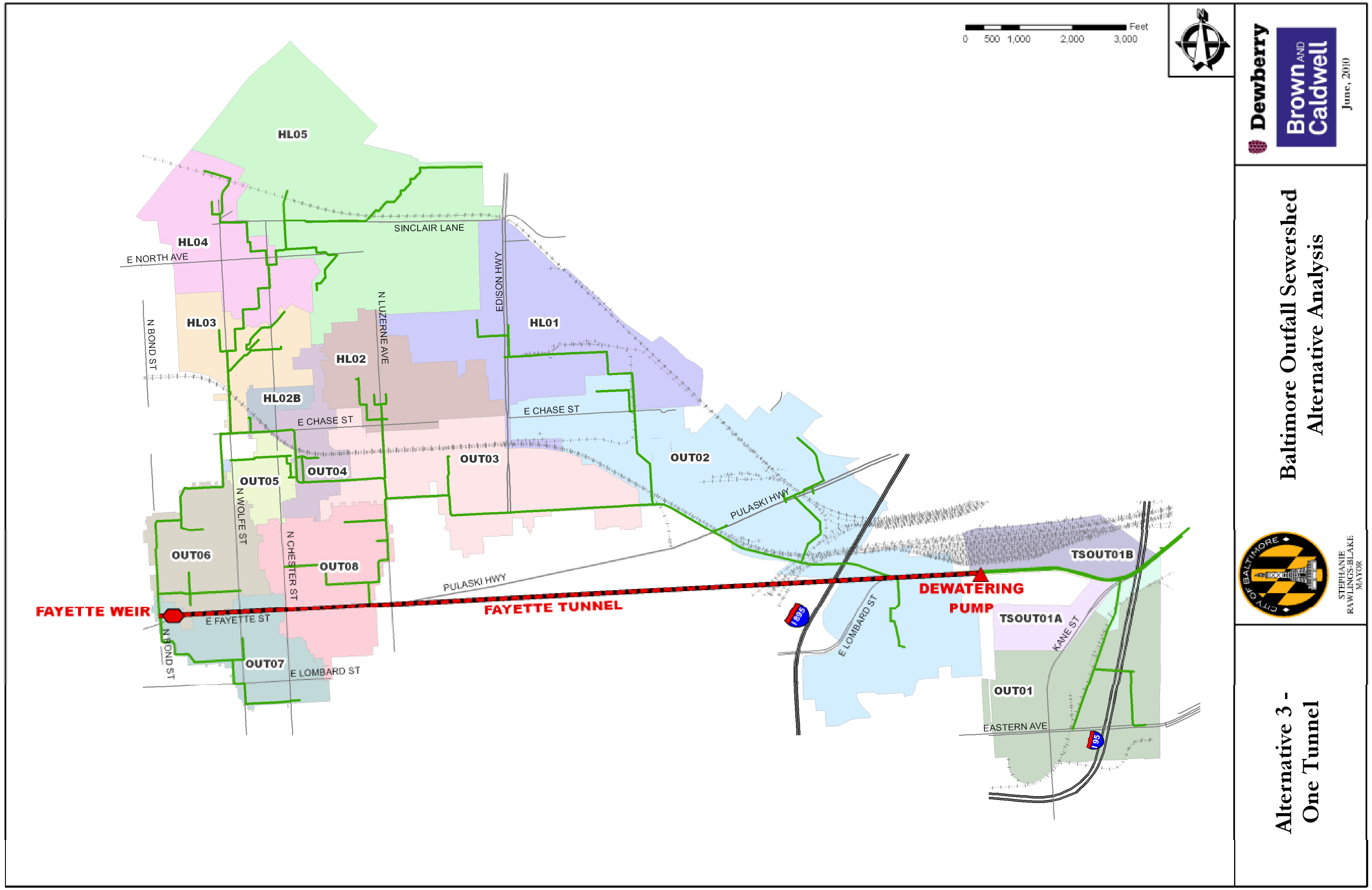
Alternative 2: Storage using One Tank, Assuming Downstream Improvements

Alternative 2 assumes that sediment is removed and downstream improvements at the Back River WWTP will accommodate higher flow rates to the plant. This alternative demonstrates the significant improvement that can be achieved in system performance due to downstream improvements. Assuming that the cleaned pipes have a Manning's roughness value of 0.013, the additional conveyance in the Outfall Interceptor is sufficient to manage the 2-year event without simulated overflows. No new storage at either the Chase or Fayette weir sites is required for the 2-year event. In the larger events, only one storage tank at the Fayette weir location is necessary.

Alternative 3: Storage-Conveyance Tunnel, Assuming Downstream Improvements

Alternative 3 uses a tunnel instead of a storage tank to protect against overflows. The tunnel starts at the Fayette weir location and generally runs in the same west to east direction as the Outfall Interceptor, although it would be a few blocks south. The flow in the tunnel re-enters the Outfall Interceptor along Lombard Street where the Outfall Relief sewer runs parallel to the Outfall Interceptor. In the model, the tunnel connection is near the location where the Dundalk sewer connects to the Outfall Interceptor. Initially, during an event the tunnel provides inline storage volume. After filling and surcharging, the tunnel flows like an inverted siphon to convey flow to the downstream connection point. After an event, the tunnel would be dewatered by a small pump. The tunnel can be seen as an upstream extension of the Outfall Relief sewer. Instead of running immediately parallel to the Outfall Interceptor, the tunnel extends the relief directly to the Fayette Weir location where relief is needed to protect the 99-inch sewer from high pumping rates from the Eastern Avenue Pump Station. By diverting excess flow into the tunnel at the Fayette weir, both the 99-inch sewer and the Outfall Interceptor are protected from overflows.

A significant benefit of the tunnel alternative is that it provides an alternative, parallel flow path to the existing Outfall Interceptor. In the same way that the Outfall Relief sewer provides supplemental conveyance capacity (and in dry weather, a redundant flow path) to the Outfall Interceptor along Lombard St, a relief tunnel would provide an alternative parallel flow path to the upstream section of the Outfall Interceptor. The upstream section of the Outfall Interceptor is a critical link in the overall conveyance system. A major incident that impairs the conveyance capacity of the existing Outfall Interceptor would have a large impact on the City. Major repairs and rehabilitation of the century-old Outfall Interceptor would be much easier to accommodate with a tunnel to serve as a redundant flow path.



Map 5.4.1.2 Alternative 3 Facilities: Storage/Conveyance Tunnel

Comparison of Alternatives:

Table 5.4.1 presents the required storage volumes at the Fayette and Chase weir locations for Alternative 1, 2 and 3 to provided protection from overflows for the 2, 5, 10, 15, and 20-year return period design storms. These results assume nominal roughness conditions (0.013) after sediment cleaning.

Table 5.4.1
Trunk Sewer SSO Alternatives
Storage Volumes (MG)

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Fayette Weir Storage Tank	3.0	7.0	10.5	12.5	14.1
	Chase Weir Storage Tank	3.3	8.1	12.2	14.5	16.5
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Storage Tank	0	2.1	4.2	5.5	6.5
Alternative 3 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Tunnel Siphon Mode	0	1.6	2.5	3.6	3.6

Figure 5.7 shows the storage volumes required for the various alternatives as a function of the storm return periods. Also shown in the figure is the simulated SSO volume with future conditions and upstream improvements. Alternatives are compared to the SSO volume caused by the upstream improvements (dark blue curve in Figure 5.7).

With sediment remaining in the pipes and no downstream improvements, the required storage volume to prevent SSOs is greater than the initial SSO volume. This case is like Alternative 1 (but with sediment remaining) and is shown by the upper gold colored curve in Figure 5.7.

The storage volume required for Alternative 1 is substantially less than the initial SSO volume because of the removal of sediment (orange curve in Figure 5.7). Sediment removal is particularly helpful in all of the alternatives because more of the flow can be conveyed by the existing trunk sewers and less volume needs to be diverted at the Fayette weir.

Downstream improvements at the Back River WWTP are complimentary to sediment removal. The lower blue curve in Figure 5.7 shows the simulated SSO volume if no new facilities are added to the Outfall sewershed and the only actions are the removal of sediment and the downstream improvements. In this case, there are no simulated overflows for the 2-year event and only 3% of the initial SSO volume remains in the 10-year event.

Alternatives 2 and 3 are only needed for the 5-year and larger events. Alternative 2 requires a tank volume that is relatively small (4.2 MG for the 10-year event). Alternative 3 requires a tunnel volume that is even smaller (2.5 MG for the 10-year event in the form of a 5-foot diameter tunnel).

The results shown in Figure 5.4.1.3 emphasize the effectiveness of downstream improvements and sediment removal. Most of the initial SSO volume is removed with those two technologies. A storage tank or a conveyance tunnel is necessary to fully remove the simulated SSO volume and to provide a greater degree of flexibility and robust performance.

The performance of these alternatives is contingent upon adequate treatment capacity at the Back River WWTP. In the Outfall Sewershed model, the assumption of adequate treatment capacity corresponds to elevated peak flow rates at the County Line. Figure 5.4.1.4 is a companion to Figure 5.4.1.3. Figure 5.4.1.3 shows the sum of the peak flows at the County Line (which is the sum of the flow in the Outfall Interceptor and the Outfall Relief sewer). In the baseline simulations, the sum of the peak flows is just less than the existing treatment capacity of 300 MGD. In the alternatives, particularly Alternative 3 with the tunnel, the sum of peak flows is between 400 and 500 MGD at the County Line. This does not include additional flow from Baltimore County. Therefore, these alternatives assume approximately 100 to 200 MGD of additional treatment capacity.

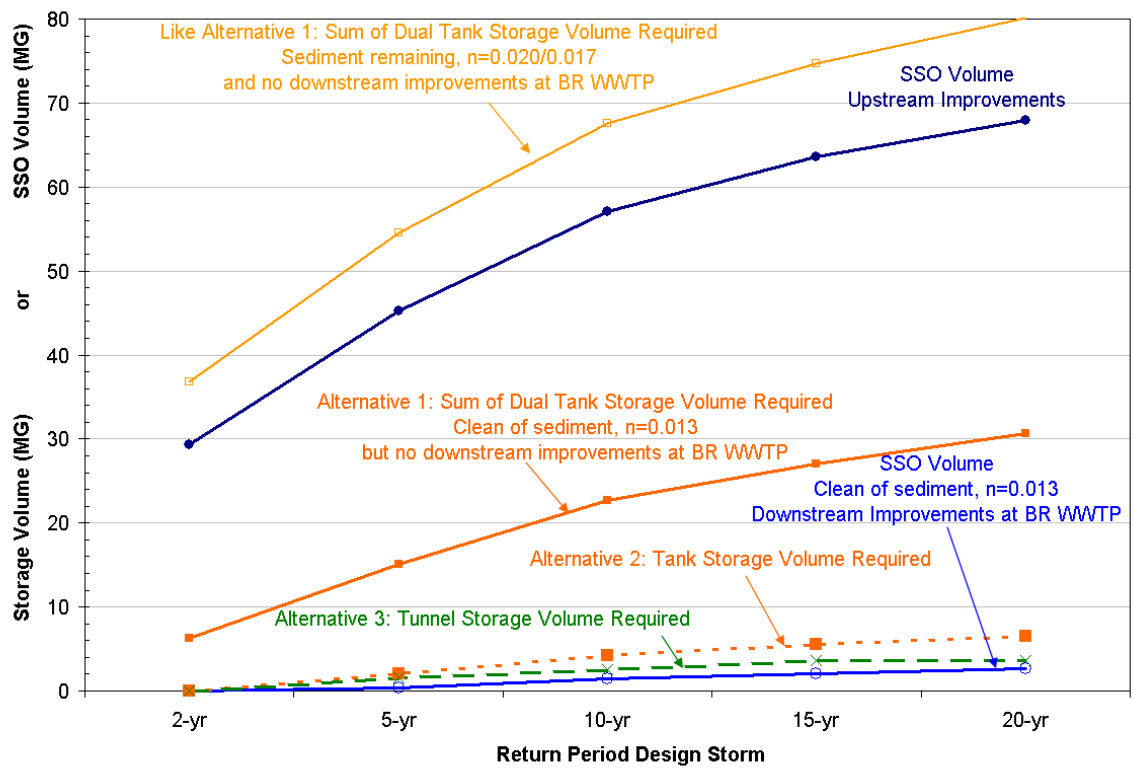


Figure 5.4.1.3 - Alternative Storage Volumes and Baseline SSO Volume

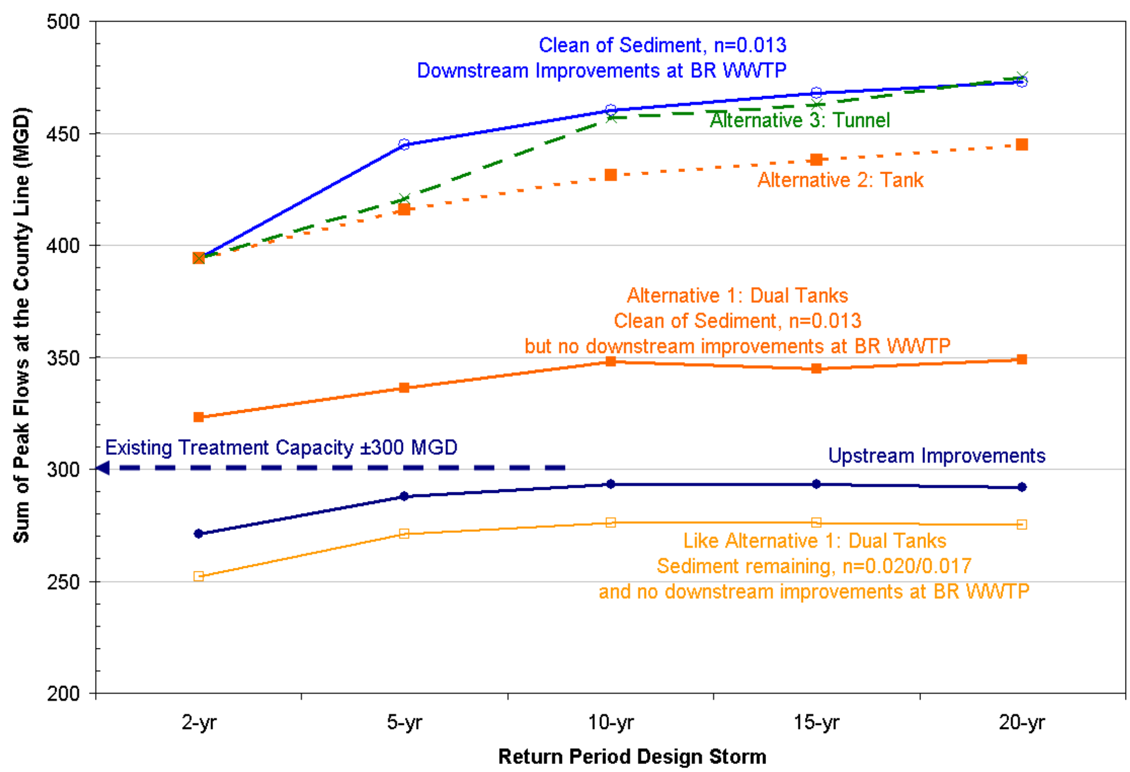


Figure 5.4.1.4 - Sum of Peak Flows at the County Line for Alternatives

Alternative Facilities Evaluated for Sub-Optimal Conditions and Large Wet Weather Events

This section is a discussion of the performance of the 10-year solutions for Alternatives 2 and 3 given above. The 10-year Alternative 2 solution is a 4.2 MG tank. The 10-year Alternative 3 solution is a 5-foot tunnel (2.5 MG volume). The facilities identified for the 10-year event are sized assuming the nominal simulation conditions (roughness of 0.013). In this analysis the performance of the facilities is evaluated for more extreme events and for a higher roughness assumption.

Simulations were run for sub-optimal conditions and larger events to evaluate the robustness of each case. For the purpose of this evaluation, “sub-optimal” conditions are defined to be a Manning’s roughness value of 0.015 and all pumps online at the Eastern Avenue Pump Station.

Figure 5.4.1.5 shows the simulated SSO volume for four cases:

- Upstream Improvements (initial SSO volume)
- Downstream Improvements and Sediment Removed (n=0.015)
- Alternative 2 (4.2 MG storage tank)
- Alternative 3 (5-foot diameter tunnel)

The improvements at the Back River WWTP make the single greatest reduction in SSO volume. Even under sub-optimal conditions, in the 2-year event, only 1% of the SSO volume remains due to the additional capacity of the downstream improvements. In the 20-year event, only 10% of the initial SSO volume remains.

For sub-optimal conditions, Alternative 2 (the 4.2 MG tank) eliminated simulated SSOs for the 2-year event and only 7% of the initial SSO remains in the 20-year event.

For sub-optimal conditions, Alternative 3 (the 5-foot diameter tunnel) eliminated the simulated SSOs for the 2-year event and only 2% of the initial SSO volume remains for the 20-year event. This result assumes that the downstream improvements allow the higher peak flows to be conveyed successfully to the Back River WWTP without surcharging at the County Line.

Both the tank and the tunnel provide significant protection for SSOs in the extreme events (15 and 10-year events), but the tunnel is more effective in minimizing overflows due to its ability to convey excess flow throughout the storm duration. A tunnel would also be more effective than a tank in back-to-back wet weather events because it does not rely on dewatering to restore the functionality of the facility.

These simulation results indicate that a facility sized for a 10-year event with nominal conditions is likely to provided protection against SSOs for a 2-year event in sub-optimal conditions.

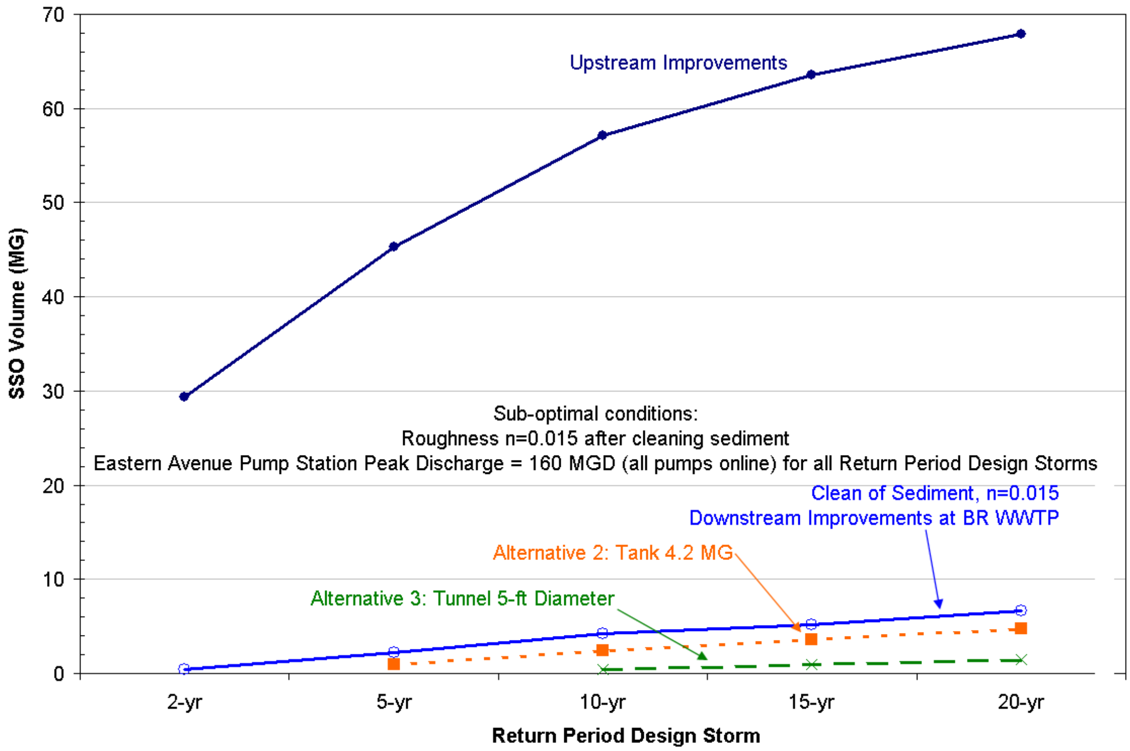


Figure 5.4.1.5 - Simulated SSO Volume for Alternatives in Sub-Optimal Conditions

Figure 5.4.1.6 shows the sum of peak flows at the County Line for the Outfall Interceptor and the Outfall Relief sewer. In the Upstream Improvements simulations, the sum of peak flows is less than 300 MGD in any event. The Outfall Sewershed alternatives assume downstream improvements at the Back River WWTP so that greater flows and lower water levels are possible at the County Line. The alternative simulations assume additional treatment capacity is sufficient to allow the flow at the County Line to increase approximately 100 MGD more than the existing rate in the 2-year event.

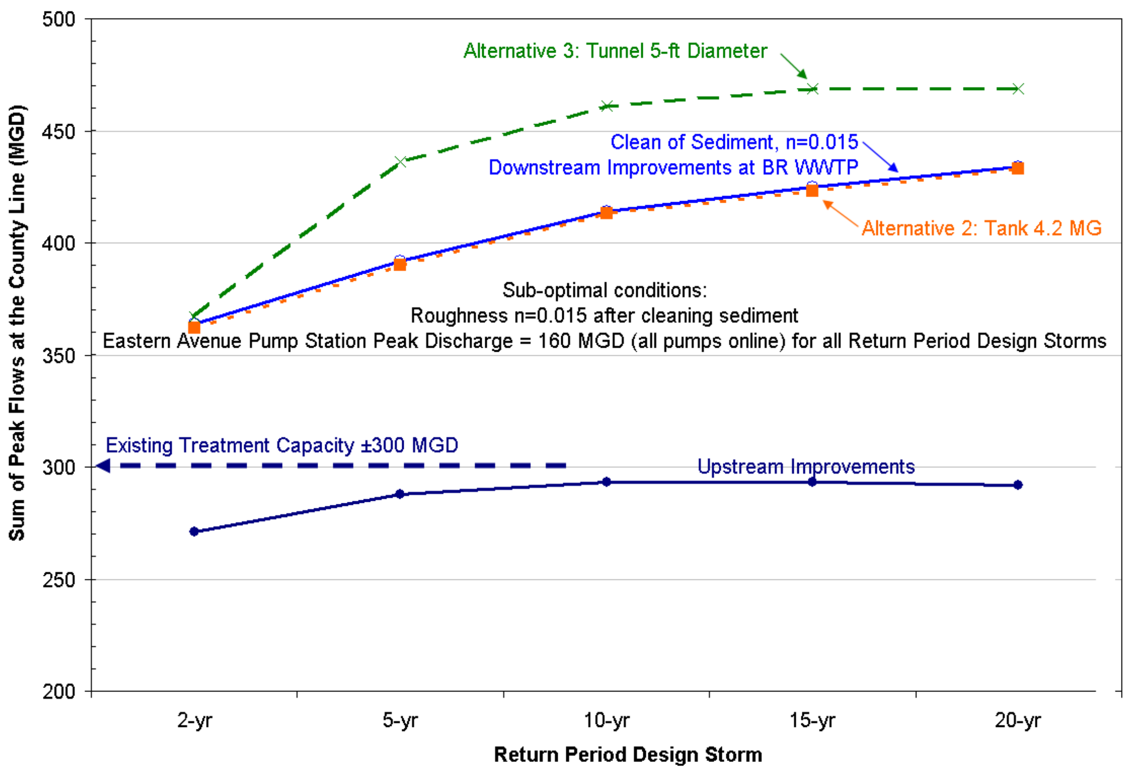


Figure 5.4.1.6 - Sum of Peak Flows at the County Line for Alternatives in Sub-Optimal Conditions

5.4.2 Summary of Improvements

Downstream improvements at the Back River WWTP and the removal of sediment from the sewers are the most effective changes to improve system performance and reduce the likelihood of overflows.

The sewershed plan for the Low Level sewershed does not mention storage to limit peak flows from the Eastern Avenue Pump Station (EAPS). Further investigation is needed using the Macro model to evaluate the trade offs between potential improvements in the Outfall Sewershed and the Low Level Sewershed. Storage upstream of the EAPS would not take advantage of the existing peak discharge capacity of the pump station. Storage located downstream of the EAPS, in the Outfall Sewershed, is likely to be more complimentary to the existing pumping capacity. In essence, the Fayette storage tank alternative serves this role. It remains for the technical program manager, using the Macro model, to further evaluate this topic.

No additional facilities are needed for the 2-year event in the Outfall Sewershed (if the assumed Manning's roughness value is accurate and the Eastern Avenue Pump Station does not operate at full capacity, not exceeding 137 MGD). Even for sub-optimum conditions, the downstream improvements and the removal of sediment are sufficient to remove 99% of the simulated SSO volume in the 2-year event compared to the initial overflow volume with Upstream Improvements.

A moderately sized storage tank or tunnel is needed at the Fayette relief point to fully eliminate SSOs for events greater than the 2-year storm and for sub-optimal conditions. Rather than defining a specific alternative recommendation, the findings of this evaluation and the summary cost tables below are presented for the purpose of discussion with the City. The cost of Alternative 2 (storage tank) is lower than the cost of Alternative 3 (tunnel). Therefore, Alternative 2 is the lowest cost approach to eliminating SSOs in the Outfall Sewershed.

Even though Alternative 3 (tunnel) is not the lowest cost option, it does provide greater flexibility and is more effective in reducing SSO volume for larger events. The advantages of a tunnel include:

- Relief for the 99-inch sewer when the Eastern Avenue Pump Station operates with all pumps on-line
- Effective reduction of SSO volume in extreme events (approximately 1 to 2% of initial SSO volume remaining)
- Functional in back-to-back wet weather events because siphon mode operation does not require dewatering time like a storage tank
- Parallel/redundant flow path to the Outfall Interceptor (useful as a dry weather bypass if the Outfall Interceptor needs maintenance, cleaning, or repair).

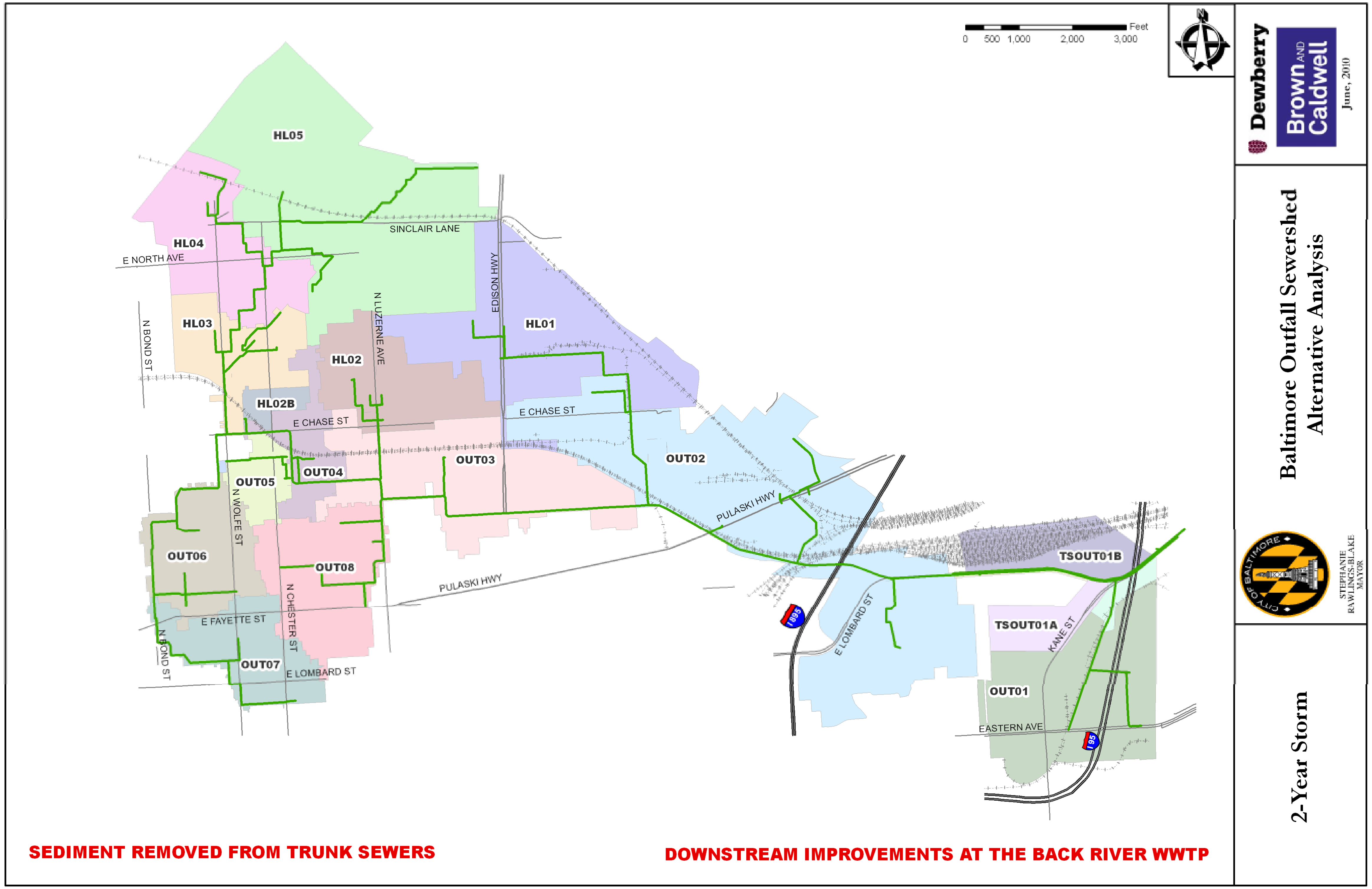
The improvements needed for each of the design storms are summarized below for Alternatives 2 and 3 for the nominal conditions. The tables presented in the summaries below itemize the recommended improvements and the costs to implement each improvement. The costs are given for 10 years (which is the span of potential implementation of the projects), from 2008 (the cost "base year") to 2017, escalated by 7% a year, as required by the methodology described in BaSES Manual, Section 8.3.2.1.

2-Year Improvements

Map 5.4.2.1 shows a summary of the improvements for the 2-year return period event. Sediment cleaning in the large diameter trunk sewers is needed along with downstream improvements.

Costs of the 2-year improvements are itemized in Table 5.4.2; the only cost in the Outfall Sewershed is the cost of removing the sediment. The cost of the downstream improvements is not included in this report. The cost of downstream improvements must include the cost of cleaning of the trunk sewers from the County Line to the Back River WWTP and the cost of storage or capacity upgrades at the treatment plant.

Table 5.4.2 2-year Outfall Improvements Alternative 3: Sediment Removed				
Site	**Improvement**	**Unit Cost**	**Quantity**	**Cost**
Sediment Cleaning in Trunk Sewers				
99-inch Sewer	Sediment Cleaning	500 $/ton	1,600 tons	$800,000
Outfall Interceptor	Sediment Cleaning	500 $/ton	29,000 tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500 $/ton	3,600 tons	$1,800,000
			Subtotal	**$17,100,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)				$7,182,000
2008 Total Estimated Cost				**$24,282,000**
2009 Total Estimated Cost				**$25,982,000**
2010 Total Estimated Cost				**$27,801,000**
2011 Total Estimated Cost				**$29,747,000**
2012 Total Estimated Cost				**$31,829,000**
2013 Total Estimated Cost				**$34,057,000**
2014 Total Estimated Cost				**$36,441,000**
2015 Total Estimated Cost				**$38,992,000**
2016 Total Estimated Cost				**$41,721,000**
2017 Total Estimated Cost				**$44,641,000**



Map 5.4.2.1 2-year Improvements for Alternative 3

5-Year Improvements

A 4-foot diameter (1.6 MG) tunnel at the Fayette site is needed in the 5-year event along with sediment removal and downstream improvements at the Back River WWTP. Branch sewer improvements are needed in meter basins HL04 and OUT01. The 5-year improvements are shown in Map 5.4.2.2.

Peak flows surcharge the sewers for the entire length of meterbasins HL03 and HL04 from the upstream end (north of Sinclair Lane) to the downstream connection at the Outfall Interceptor (at Wolfe Street and Chase Street). There is a risk of SSOs at several locations along this sewer system where the maximum HGL approaches the ground surface. Overflows are most likely at manhole S45OO_014MH (Wolfe Street and Darley Street) because of a low ground surface elevation at this point (less than 4 feet of cover). The SSO location is active for the 5-year and larger events.

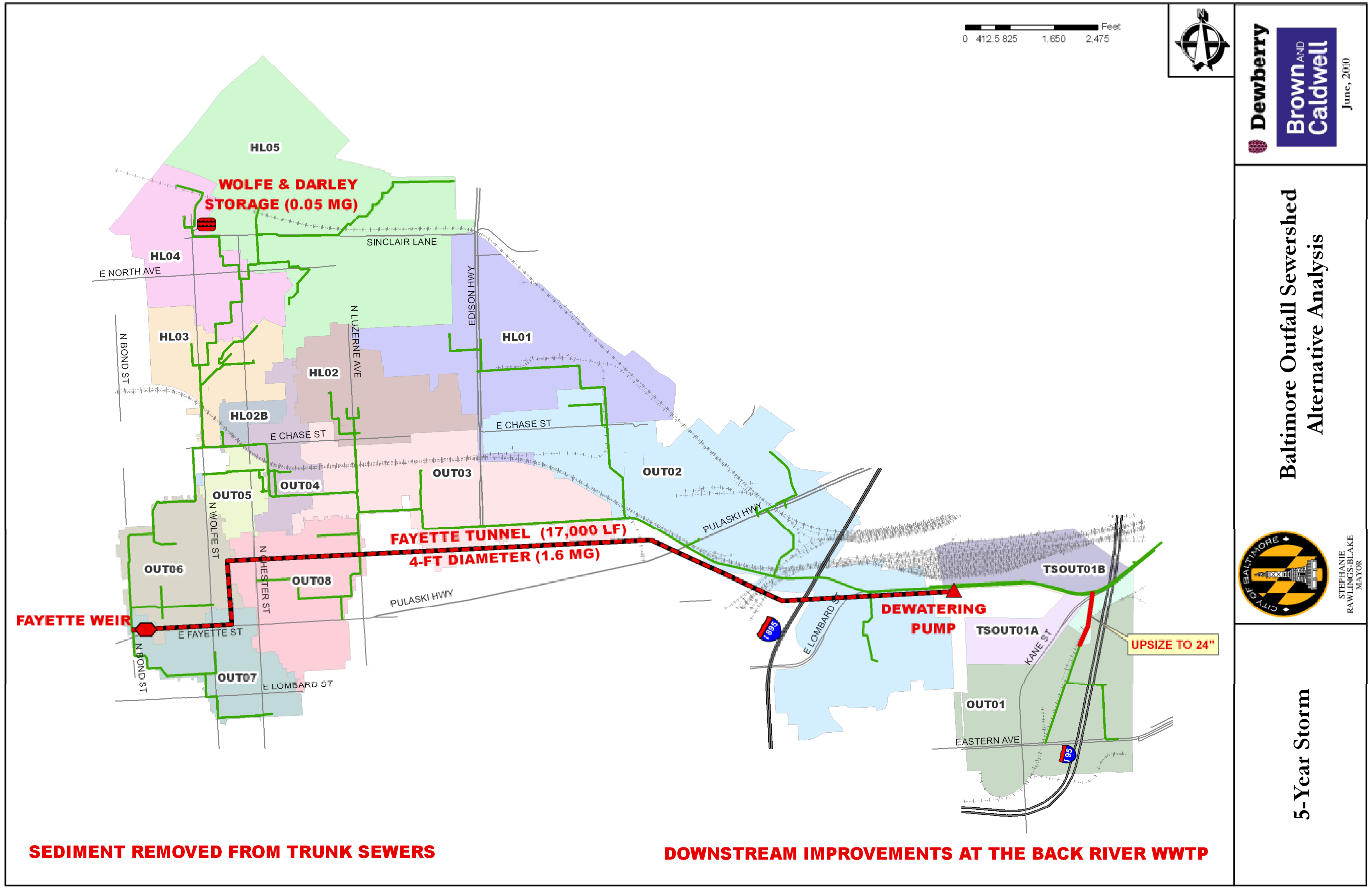
Possible solutions include sealing the manhole, raising the manhole rim to an elevation that is similar to neighboring manholes (approximately 3 feet), building a small storage tank, or rehabilitation of sewers in the Darley/Cliftview Avenue neighborhood to reduce infiltration and inflow (I/I). A storage tank alternative or sewer rehabilitation to reduce I/I will reduce peak flows to the downstream pipes leading to the Outfall Interceptor, thus decreasing the risk of SSO at other locations which do not have simulated SSOs but are at risk of SSOs due to high water levels.

The 18-inch sewer serving meterbasin OUT01 runs along the railroad tracks parallel to and between Kane Street and the Interstate-95 freeway. There is one simulated SSO location in the lower section of the pipe for the 5-year and larger events. The simulated SSO is caused by high simulated peak flows that exceed pipe capacity. The volume of the SSO increases when the Outfall Interceptor is surcharged, but this downstream surcharge condition is not the primary cause of the SSO. The alternative solution is a 24-inch sewer replacement, running 1012 LF from manhole S69C__002MH to the connection to the Outfall Interceptor at manhole S71A__007MH.

Costs of the 5-year improvements are itemized in Table 5.4.3.

Table 5.4.3
5-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.047	MG	$282,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 4' x 17,000 LF	44.14	$/gal	1.6	MG	$70,533,060
	Dewatering Pump	3.00	$/gpd	2	MGD	$6,000,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$95,008,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$39,903,000
2008 Total Estimated Cost						**$134,911,000**
2009 Total Estimated Cost						**$144,355,000**
2010 Total Estimated Cost						**$154,460,000**
2011 Total Estimated Cost						**$165,272,000**
2012 Total Estimated Cost						**$176,841,000**
2013 Total Estimated Cost						**$189,220,000**
2014 Total Estimated Cost						**$202,465,000**
2015 Total Estimated Cost						**$216,638,000**
2016 Total Estimated Cost						**$231,803,000**
2017 Total Estimated Cost						**$248,029,000**



Map 5.4.2.2 5-year Improvements for Alternative 3

10-Year Improvements

For 10-year level of protection, a 5-foot tunnel (2.5 MG) is required at the Fayette relief point. The 10-year improvements are shown in Map 5.4.2.3.

Additional branch sewer improvements are needed in meterbasin HL05 and OUT01. In the HL05 meterbasin, there is a simulated SSO along Sinclair Lane at Homestead Street (manhole S47MM_042MH) for the 10-year and larger events. This manhole is vulnerable to overflow because of a downstream hydraulic restriction along Collington Avenue. The size of the pipe along Collington Avenue needs to be increased from 12 to 15-inches to eliminate the SSO further upstream at Sinclair and Homestead. The 15-inch replacement pipe would run 592 LF along Collington Avenue from manhole S47MM_031MH (Sinclair & Collington) to manhole S45MM_025MH (in an alley west of Collington Avenue and north of North Avenue).

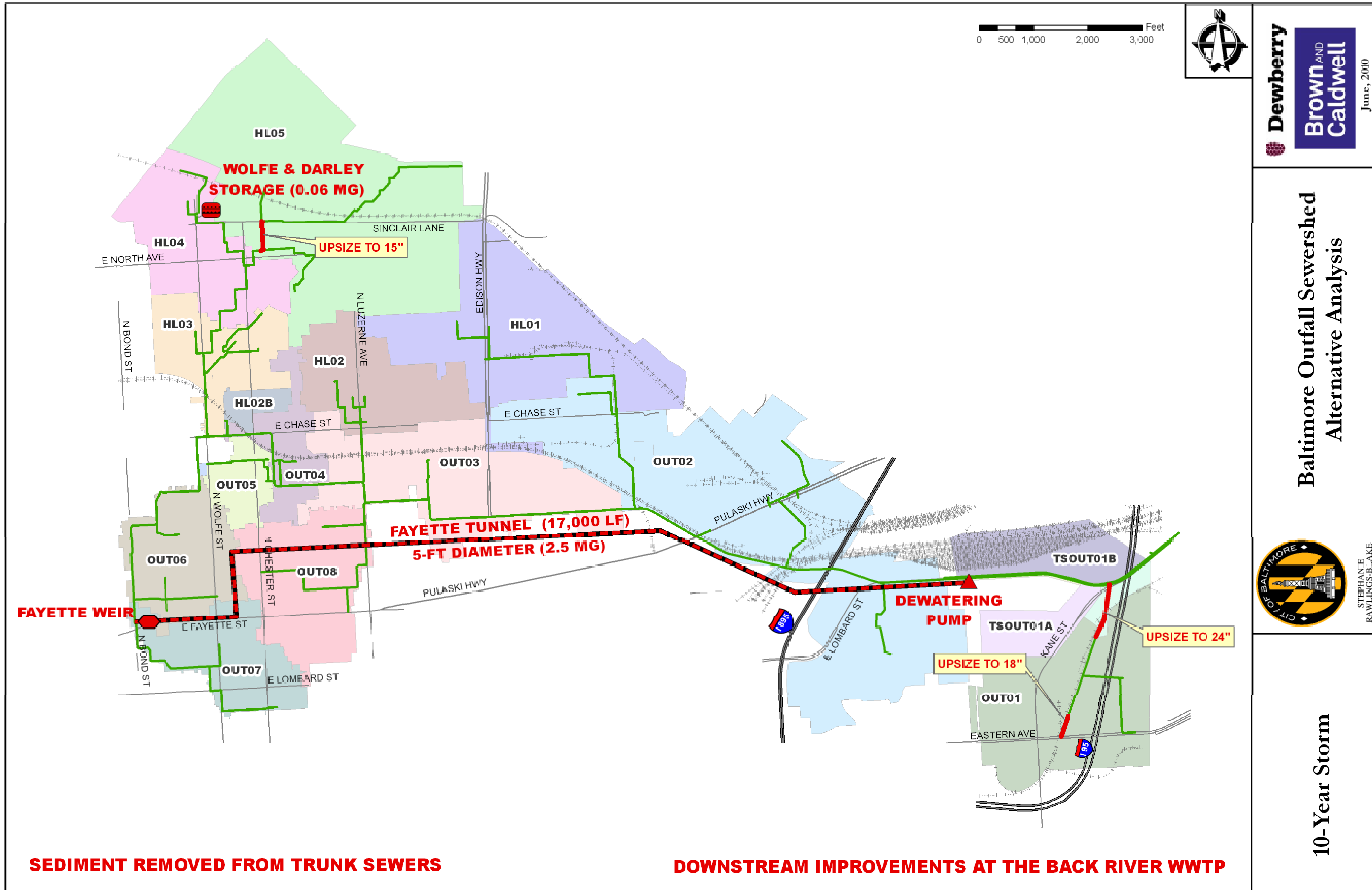
In the OUT01 meter basin, manhole S69G__005MH is the upstream end of the upper section in the model. This manhole, at Eastern Avenue, is the location of a small simulated overflow for the 10-year and larger events. The first pipe section in the model is a 15-inch pipe; all of the other pipe sections along this branch sewer are 18-inch diameter. The 10-year event requires a replacement pipe running approximately 400 LF from manhole S69G__005MH (at Eastern Avenue) to the next manhole north, S69G__008MH. The replacement pipe is upsized from 15 to 18-inches

Costs of the 10-year improvements are itemized in Table 5.4.4.

Based on the results of the sensitivity analysis for sub-optimal conditions in the AARR, the facilities needed for a 2-year level of protection in sub-optimal conditions are equivalent to those needed for the 10-year event with nominal conditions. Thus the major facilities costs presented in Table 5.4.4 are representative of the cost of facilities for a 2-year level of protection under sub-optimal conditions. These facilities are robust and provide protection with a greater degree of certainty. Even in extreme events greater than 10-year recurrence, these facilities are very effective in reducing the volume of SSOs, even if compete protection is not achieved.

Table 5.4.4
10-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.065	MG	$390,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
OUT01 Upper Section	18" Replacement Pipe	585	$/LF	400	LF	$234,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 5' x 17,000 LF	31.65	$/gal	2.5	MG	$79,023,110
	Dewatering Pump	2.84	$/gpd	2.5	MGD	$7,100,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$105,286,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$44,220,000
	2008 Total Estimated Cost					**$149,506,000**
	2009 Total Estimated Cost					**$159,971,000**
	2010 Total Estimated Cost					**$171,169,000**
	2011 Total Estimated Cost					**$183,151,000**
	2012 Total Estimated Cost					**$195,972,000**
	2013 Total Estimated Cost					**$209,690,000**
	2014 Total Estimated Cost					**$224,368,000**
	2015 Total Estimated Cost					**$240,074,000**
	2016 Total Estimated Cost					**$256,879,000**
	2017 Total Estimated Cost					**$274,861,000**



Map 5.4.2.3 10-year Improvements for Alternative 3

15-Year Improvements

For 15-year level of protection, a 6-foot tunnel (3.6 MG) is required at the Fayette relief point. The 15-year improvements are shown in Map 5.4.2.4.

Branch sewer facilities added for the 15-year level of protection include a second small storage tank and a replacement sewer in the HL04 meter basin. The storage tank is needed in the vicinity of North Avenue and Chester Street to reduce peak flows to the downstream sections of pipe. Not only do the larger events require additional storage at the Wolfe and Darley location, but 554 LF of pipe along Wolfe Street and Darley Street need to be upsized from 10 to 12 inches.

In meter basin HL05, the 12-inch sewer along Sinclair Lane needs to be upsized to 15-inches. This segment is 751 LF from manhole S47MM_042MH (Sinclair and Homestead) to Collington Avenue at manhole S47MM_031MH. In the upper section of meter basin OUT01, the 15-year event requires 1600 LF of pipe upsized to 21 inches from manhole S69G__005MH to manhole S69E__005MH. Costs of the 15-year improvements are itemized in Table 5.4.5

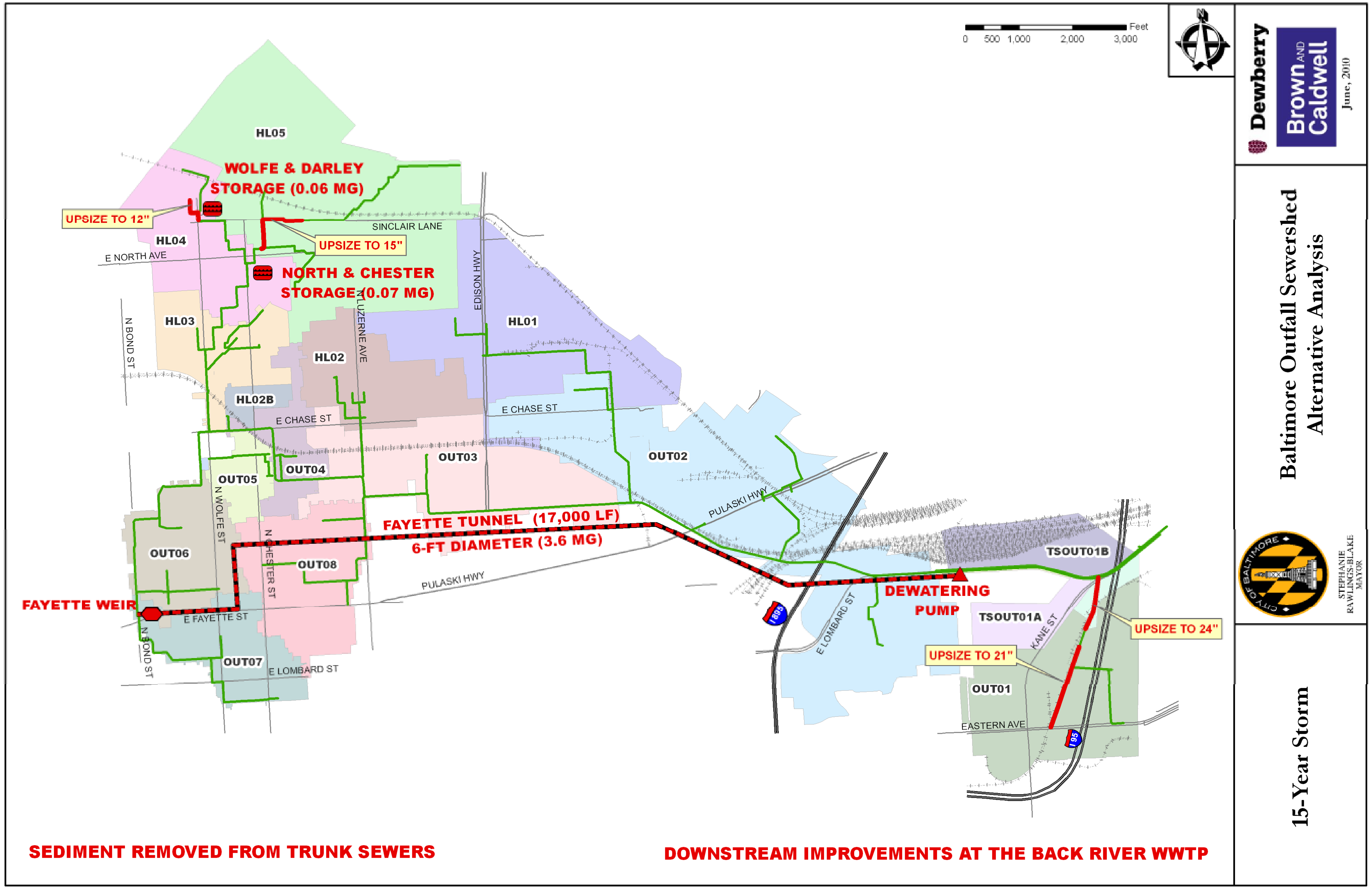
If a RDII reduction alternative were to be used instead of a storage tank, the peak flows from the Darley/Cliftview Avenue neighborhood would need to be reduced 30 to 50%. More extensive RDII reduction would be needed to provide the same benefit at the North and Chester storage tank.

The cost of RDII reduction was investigated. The Darley/Cliftview neighborhood has approximately 11,000 LF of sewers ranging in size from 8 to 24 inches. The cost to rehabilitate these sewers to reduce RDII would be approximately $3 million.

RDII reduction in the sewers upstream of the North/Chester overflow location would require rehabilitation of approximately 20,000 LF of pipe with a cost of $5.5 million. The total cost of RDII in the HL04 meter basin area would be approximately $8.5 million. The cost of RDII reduction is approximately an order of magnitude more than the cost of storage tanks, not considering the cost to convey and treat the extraneous RDII flow.

Table 5.4.5
15-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
			Branch Sewer Improvements			
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.058	MG	$348,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.073	MG	$438,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
			Major Relief Facilities			
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
			Sediment Cleaning in Trunk Sewers			
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$115,909,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,682,000
		2008 Total Estimated Cost				**$164,591,000**
		2009 Total Estimated Cost				**$176,112,000**
		2010 Total Estimated Cost				**$188,440,000**
		2011 Total Estimated Cost				**$201,631,000**
		2012 Total Estimated Cost				**$215,745,000**
		2013 Total Estimated Cost				**$230,847,000**
		2014 Total Estimated Cost				**$247,006,000**
		2015 Total Estimated Cost				**$264,296,000**
		2016 Total Estimated Cost				**$282,797,000**
		2017 Total Estimated Cost				**$302,593,000**



Map 5.4.2.4 15-year Improvements for Alternative 3

20-Year Improvements

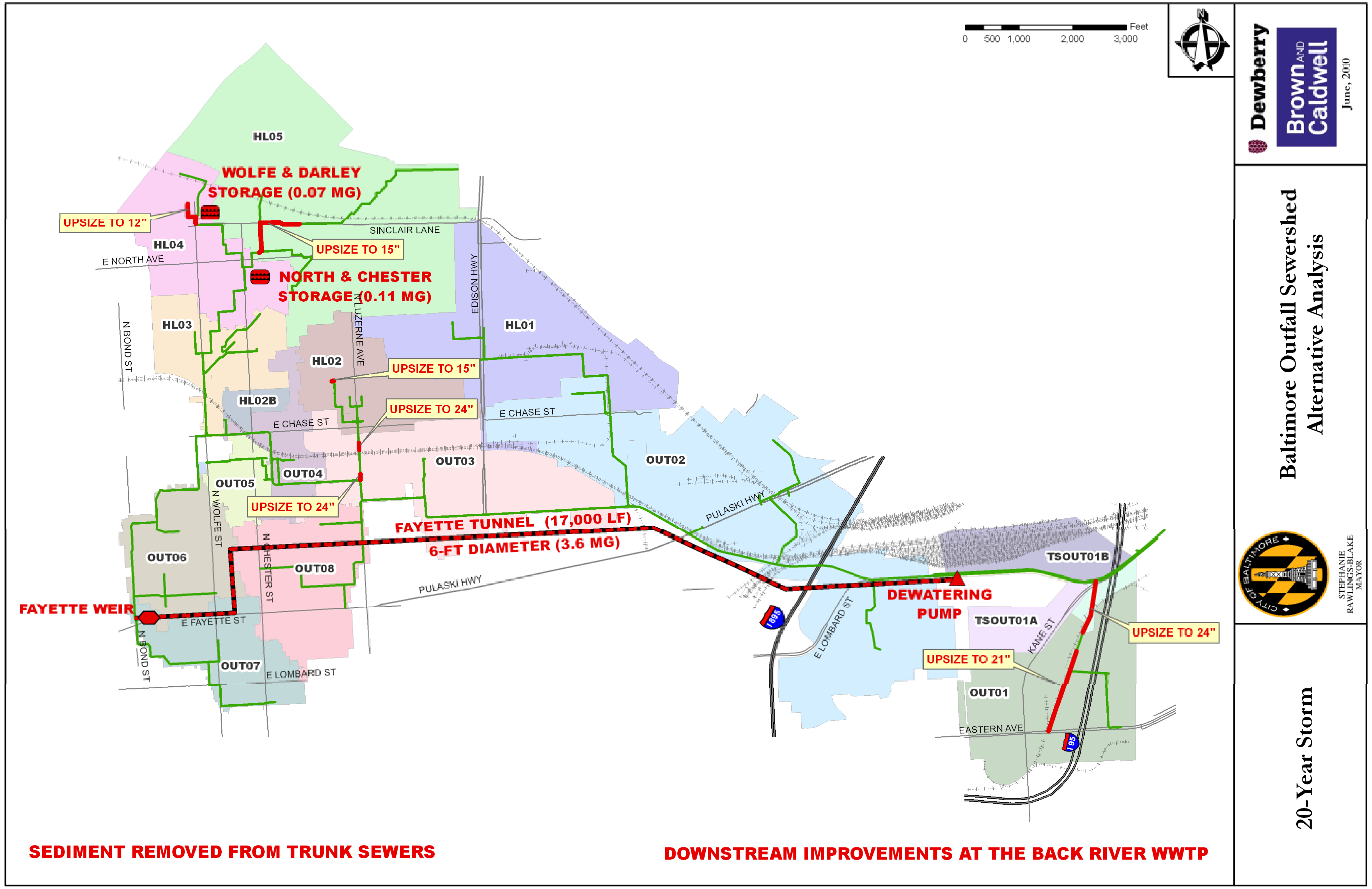
In general, the facilities needed for the 20-year event are very similar to those needed for the 15-year event. For 20-year level of protection, a 6-foot tunnel (3.6 MG) is required at the Fayette relief point. The 6-foot diameter tunnel, size for the 15-year level of protection, is also adequate to provide a 20-year level of protection. The 20-year improvements are shown in Map 5.4.2.5.

In the 20-year event, high peak flow rates cause surcharging all along the length of the HL02 branch sewer. To eliminate the SSO, upsizing the pipe near the downstream end of the branch sewer is recommended. The replacement pipes along Luzerne Street require upsizing from 15 inches to 18 inches. The first segment of the replacement runs 134 LF from manhole S49EE_004MH (Beryl Street) to manhole S49EE_021MH. The second segment of the replacement runs 137 LF from manhole S49CC_021MH to manhole S49CC_075UN (Ashland Street at the connection to the Outfall Interceptor). The total length of replacement along Luzerne Street is approximately 271 LF.

At the upstream end of the HL02 branch in the model there is a small overflow at Milton Street north of Preston Street (manhole S49GG_039MH) in the 20-year event. The short 10-inch sewer that crosses under the road needs to be upsized to 15 inches for 46 LF from manhole S49GG_039MH to manhole S49GG_027MH.

Costs of the 20-year improvements are itemized in Table 5.4.6.

Table 5.4.6 20-year Outfall Improvements Alternative 3: Tunnel, Sediment Removed						
Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL02 Milton Ave	15" Replacement Pipe	585	$/LF	46	LF	$26,910
HL02 Luzerne St	24" Replacement Pipe	1080	$/LF	271	LF	$292,680
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.074	MG	$444,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.107	MG	$642,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$116,529,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,942,000
2008 Total Estimated Cost						**$165,471,000**
2009 Total Estimated Cost						**$177,054,000**
2010 Total Estimated Cost						**$189,448,000**
2011 Total Estimated Cost						**$202,709,000**
2012 Total Estimated Cost						**$216,899,000**
2013 Total Estimated Cost						**$232,082,000**
2014 Total Estimated Cost						**$248,328,000**
2015 Total Estimated Cost						**$265,711,000**
2016 Total Estimated Cost						**$284,311,000**
2017 Total Estimated Cost						**$304,213,000**



Map 5.4.2.5 20-year Improvements for Alternative 3

Summary of Costs

Figure 5.4.2.6 shows the total costs for Alternatives 1, 2, and 3. Alternative 1 does not assume any downstream improvements at the Back River WWTP. This is the cost to manage the SSO problem within the Outfall sewershed with facilities located in the Outfall Sewershed alone. Alternative 1 does not address peak flows into the Back River WWTP that exceed the plant's existing treatment capacity.

Alternatives 2 and 3 assume that there are downstream improvements at the Back River WWTP, but the cost of those downstream improvements are not accounted for in this cost summary. The cost of Alternatives 2 and 3 are substantially lower than Alternative 1 because of the downstream improvements at the Back River WWTP. Even though the cost of Alternative 3 is greater than Alternative 2, the additional flexibility of the tunnel facilities merits consideration when choosing between the tank and tunnel concepts.

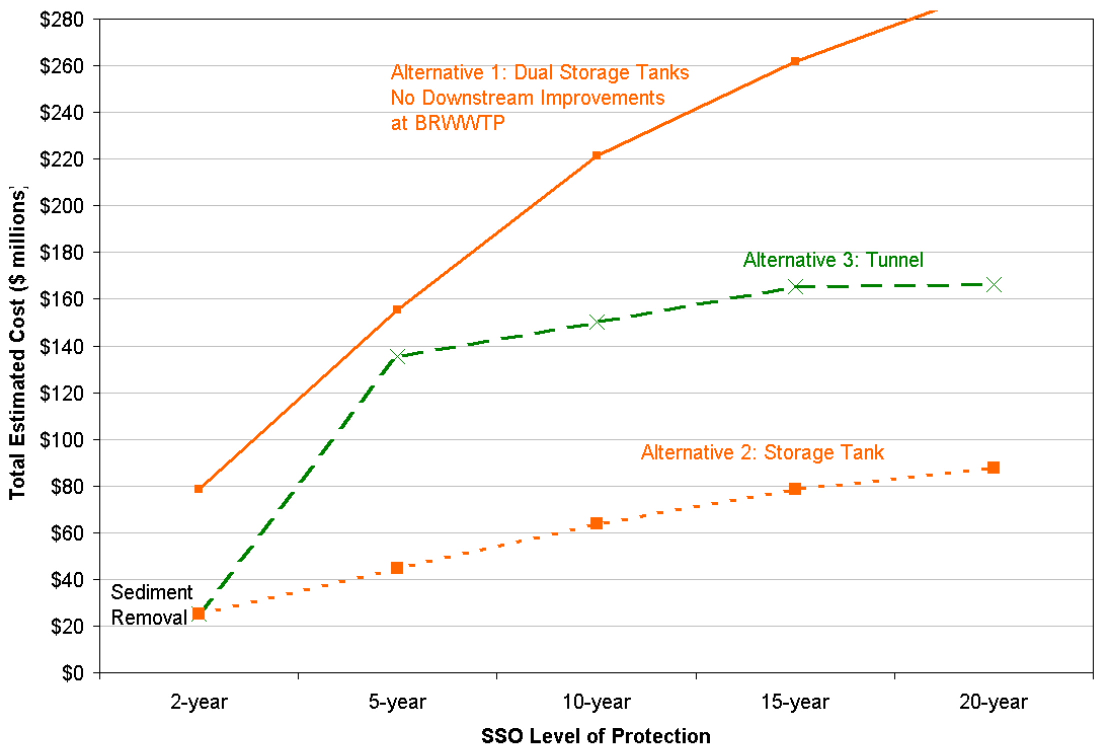


Figure 5.4.2.6 2008 Total Estimated Cost of Alternative 3

Construction costs were developed for all alternatives evaluated. To develop the estimated costs of construction, standard unit costs for sewer point repairs, sewer lining, sewer replacement, sewer cleaning, and manhole rehabilitation/replacement were provided by the City in 2008 dollars. The construction costs provided were fully loaded costs to address such items as mobilization, maintenance of traffic, paving restoration, bypass pumping and miscellaneous (non-sanitary) utility work. For costs not provided by the City (large diameter tunnels and pumping stations) recent projects within the

City and surrounding areas were reviewed to assist in estimating the most probable fully loaded cost of construction.

In addition to these construction costs, an additional 42 percent was added to accommodate engineering design, construction management/inspection, administration, post-award engineering services and contingencies. A 7 percent annual inflation rate is used to project costs for years beyond 2008.

Alternative 3 total estimated costs for the Outfall Sewershed improvements are summarized in Table 5.4.7 for the 2, 5, 10, 15, and 20-year events; the costs are inflated 7% per year for the recommended projects depending upon the year they might be implemented (from 2008 through 2017). The total estimated costs are under the column heading "Cumulative" in Table 5.4.7 for the 5, 10, 15, and 20-year events. The "Additional" cost column in the table is the incremental cost of facilities from one design storm level of protection to the next.

Table 5.4.8 is a summary of total estimated cost normalized by the volume of SSO removed. The units are dollars per gallon of SSO removed. The cumulative cost divided by the cumulative SSO volume removed is a direct normalization of the total cost by the total SSO volume. For example: The 2-year facilities removed 29.3 MG of SSO at a cost of $24 million; thus the unit cost is $0.83 per gallon of SSO removed. The 2-year facilities eliminate all of the SSOs in the 2-year event.

Incremental normalized cost values are also given in the table under the "Additional" columns. The additional costs per additional gallon of SSO volume removed were developed in the following manner: The 2-year facilities are effective in removing much of the SSO volume for the 5-year event, but the remaining SSO volume is 0.32 MG with the 2-year facilities in place. The additional cost of the 5-year facilities is $111 million compared to the 2-year facilities. The 5-year facilities are needed to remove the 0.32 MG of SSO that would remain if the 2-year facilities were in place. Therefore, the normalized additional cost is $346 per gallon of additional SSO removed.

The step wise progression was used to determine the additional SSO that could be removed by the 10-year facilities compared to the SSO remaining with the 5-year facilities. The normalized additional cost is $730 per gallon of additional SSO removed to reach the 10-year level of protection.

Likewise, the analysis determined the additional costs and the additional SSO volumes removed by the 15 and 20-year facilities. The additional volumes removed in these cases are negligible; therefore, the normalized additional costs are undefined.

The additional SSO removed is a relatively small volume because facilities sized for a smaller event are very effective at removing most of the SSO volume in a larger event, even though they may not be adequate to remove 100% of the SSO volume. As a result, the normalized costs ($/gallon) to remove the additional SSO volumes are extremely high.

Table 5.4.7
Total Estimated Outfall Improvement Costs

Projected Year	2-yr Cost	5-yr		10-yr		15-yr		20-yr	
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$24,282,000	$110,629,000	$134,911,000	$14,595,000	$149,506,000	$15,085,000	$164,591,000	$880,000	$165,471,000
2009	$25,982,000	$118,373,000	$144,355,000	$15,616,000	$159,971,000	$16,141,000	$176,112,000	$942,000	$177,054,000
2010	$27,801,000	$126,659,000	$154,460,000	$16,709,000	$171,169,000	$17,271,000	$188,440,000	$1,008,000	$189,448,000
2011	$29,747,000	$135,525,000	$165,272,000	$17,879,000	$183,151,000	$18,480,000	$201,631,000	$1,078,000	$202,709,000
2012	$31,829,000	$145,012,000	$176,841,000	$19,131,000	$195,972,000	$19,773,000	$215,745,000	$1,154,000	$216,899,000
2013	$34,057,000	$155,163,000	$189,220,000	$20,470,000	$209,690,000	$21,157,000	$230,847,000	$1,235,000	$232,082,000
2014	$36,441,000	$166,024,000	$202,465,000	$21,903,000	$224,368,000	$22,638,000	$247,006,000	$1,322,000	$248,328,000
2015	$38,992,000	$177,646,000	$216,638,000	$23,436,000	$240,074,000	$24,222,000	$264,296,000	$1,415,000	$265,711,000
2016	$41,721,000	$190,082,000	$231,803,000	$25,076,000	$256,879,000	$25,918,000	$282,797,000	$1,514,000	$284,311,000
2017	$44,641,000	$203,388,000	$248,029,000	$26,832,000	$274,861,000	$27,732,000	$302,593,000	$1,620,000	$304,213,000

Table 5.4.8
Total Estimated Outfall Improvement Costs per Gallon SSO Removed

SSO Volume (MG)	**Upstream Improvements 2-yr**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Remaining with 2-yr Facilities	Upstream Improvements	Remaining with 5-yr Facilities	Upstream Improvements	Remaining with 10-yr Facilities	Upstream Improvements	Remaining with 15-yr Facilities	Upstream Improvements
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**
SSO Volume Removed (MG)	**2-yr**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Additional SSO Removed by 5-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 10-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 15-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 20-yr Facilities	Cumulative SSO Removed
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**
Projected Year	**2-yr Cost**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$0.83	$346.00	$2.98	$730.00	$2.62	undefined	$2.59	undefined	$2.44
2009	$0.89	$370.00	$3.19	$781.00	$2.80	undefined	$2.77	undefined	$2.61
2010	$0.95	$396.00	$3.41	$835.00	$3.00	undefined	$2.96	undefined	$2.79
2011	$1.02	$424.00	$3.65	$894.00	$3.21	undefined	$3.17	undefined	$2.99
2012	$1.09	$453.00	$3.90	$957.00	$3.43	undefined	$3.39	undefined	$3.19
2013	$1.16	$485.00	$4.18	$1,024.00	$3.67	undefined	$3.63	undefined	$3.42
2014	$1.24	$519.00	$4.47	$1,095.00	$3.93	undefined	$3.88	undefined	$3.66
2015	$1.33	$555.00	$4.78	$1,172.00	$4.20	undefined	$4.16	undefined	$3.91
2016	$1.42	$594.00	$5.12	$1,254.00	$4.50	undefined	$4.45	undefined	$4.19
2017	$1.52	$636.00	$5.48	$1,342.00	$4.81	undefined	$4.76	undefined	$4.48

References

(1) National Weather Service, 2004. "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 3, G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
Depth, duration, and frequency data for Baltimore WSO City available online from the National Weather Service, Hydrometeorological Design Studies Center, Precipitation Frequency Data Server
http://dipper.nws.noaa.gov/hdsc/pfds/orb/md_pfds.html

(2) U.S. Soil Conservation Service. Technical Release 55: Urban Hydrology for Small Watersheds. USDA (U.S. Department of Agriculture). June 1986. Available from NTIS (National Technical Information Service), NTIS # PB87101580. Also available on the web in .pdf format at
http://www.info.usda.gov/CED/ftp/CED/tr55.pdf

6.0 Geographic Information System (GIS)

6.1 Overview of GIS

The City of Baltimore maintains a robust Geographic Information System (GIS) representing the wastewater infrastructure. The GIS is housed in an ESRI format Geodatabase and leverages the enterprise capabilities of ArcSDE. At the time of this report, this data was compiled using ArcGIS version 9.2. An integral part of the sewershed study is the update of the GIS to represent the existing conditions at the time of the study. These updates provided to the City were considered "Core" data deliveries as they are the primary or core repository of data representing the wastewater infrastructure. This is in comparison to "non-core" data which was the supplemental data provided to the City such as manhole inspection reports, CCTV video, etc. This section describes the City's GIS system; describes the methods and procedures used during the project to update the system; and the quality assurance procedures performed to verify the accuracy of the work performed.

The wastewater utility geodatabase is comprised of three thematic groups of features:

- <u>Lines Thematic Group</u> – contains wastewater features that can be represented as lines whose direction indicates the direction of flow. These line features make up the foundation of the wastewater network. All features in this thematic group participate in the geometric network. These features include:

    - House Connection (line)
    - Sewer (line)

- <u>Features Thematic Group</u> – contains wastewater features that can be represented as points, lines and/or polygons. The features in this thematic group do not affect flow and will not participate in the geometric network. Traces and other network analysis operations do not consider these entities, yet they are captured in the database to provide a more complete representation of the system. These features include:

    - Casing (polygon)
    - Facility (polygon)
    - Lamphole (point)
    - Manhole Cover (point)
    - Structure (polygon)

- <u>Devices Thematic Group</u> – contains wastewater features that can be represented as points. All features in this thematic group participate in the geometric network. These features include:

    - Manhole Junction (point)

- Meter Station (point)
- Pump Station (point)
- Treatment Plant (point)
- Bend (point)
- Valve (point)
- House End (point)
- House Intersection (point)
- House Sewer Intersection (point)
- Sewer End (point)
- Sewer Intersection (point)

The Outfall Sewershed consisted almost entirely of gravity systems, and therefore contained no pressure systems or related features (such as bends, pump stations, etc.).

The following graphic summarizes the feature objects in the City's wastewater GIS.

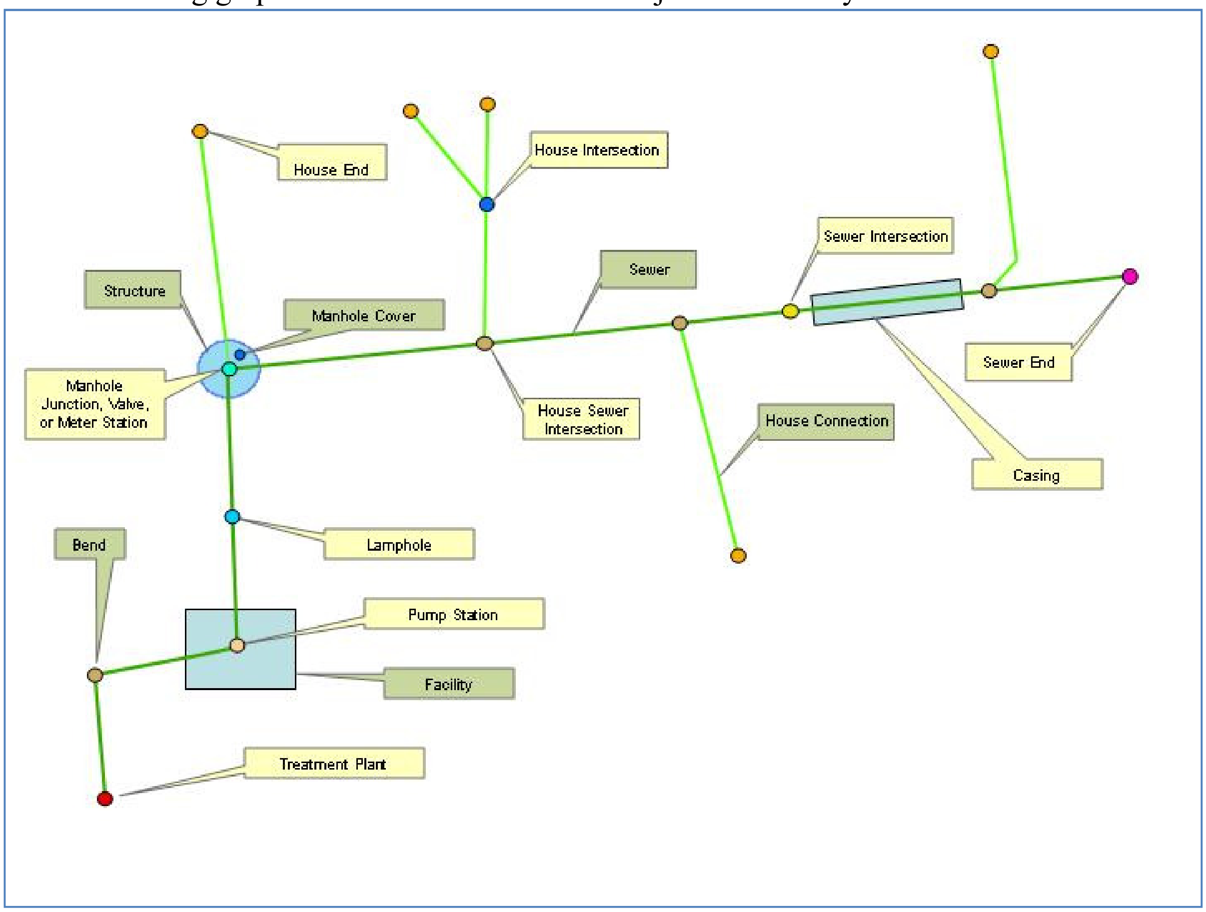


Figure 6.1 - Feature Objects in the City's Wastewater GIS

6.2 Field Data and GIS Integration

The Sewershed Study and Evaluation project involved extensive field activities which generated significant amounts of non-core data to be used to update the core GIS. Specifically, the non-core data generated was:

- Manhole Inspection Data
- GPS Survey Data
- Closed Circuit Television (CCTV) Inspection Data
- Smoke Testing
- Dyed Water Testing Data

The majority of the spatial and attribute edits made to the wastewater geodatabase were based on information extracted from these non-core datasets, namely the manhole inspections, and GPS survey data. When current conditions could not be established through these sources, additional engineering contract document research was performed to populate the GIS. The following is further description regarding the field collected data and its use in updating the GIS.

Manhole Inspections

Manhole inspections were performed on 1845 manholes in the Outfall sewershed. Information was collected using a custom designed Manhole Inspection Application Software (MIAS) application. MIAS allows field crews to collect detailed attribute information about the physical characteristics of a manhole, its sewer connections, and the manhole's surrounding environment. In addition to characteristics such as size, shape, and material, the application records the condition and infiltration properties of the manhole's features. The MIAS application captures inventory and condition information for the following manhole components:

- Location
- Environment
- Cover
- Frame
- Chimney/Stack
- Corbel
- Barrel
- Bench
- Channel
- Pipe Connections

The unique identifier used in both the GIS and MIAS datasets is the MANHOLE_ID field. This common field allowed for database joins which facilitated integration of the manhole inspection field information directly into wastewater feature attribute fields.

Roughly 11,707 manhole inspection photos were taken during the manhole inspections in the Outfall sewershed. The MIAS application and other GIS tools provided easy access to these photos for use in checking and validating the manhole information being entered into the database.

GPS Manhole Surveys

A total of 1,811 survey-grade GPS survey locations of manhole covers were completed during the project, representing 93% of all City-owned manholes. The remaining manholes were not surveyed due to access issues. These GPS locations were used to position key manhole features and to establish the rim elevation stored in the manhole cover GIS feature class.

The GPS rim elevations were used along with depths measured during the manhole inspection, from the rim down to the invert of each pipe connecting manhole, to establish pipe invert elevations in the Sewer feature layer.

Rim elevations for manholes that were not GPS surveyed were extracted from construction drawings where available. If rim elevations were not available on the contract drawing, the raw invert elevations from the construction drawings were used, with those elevations being converted from the City's vertical datum to NAVD88 using the provided conversion factor.

CCTV Inspections

The Outfall sewershed study plan team completed roughly 2,107 individual CCTV sewer inspections. The up and down nodes for each CCTV survey were verified that they link to a valid GIS manhole, lamphole or SewerEnd features that represent the starting and ending locations of the survey.

With the data relationship established, the CCTV surveys, manhole inspections (MIAS database) and the GIS were compared to assist in GIS attribute updating.
The CCTV surveys were invaluable in the GIS updating process by enabling Engineers and GIS technicians to:

- Locate previously unknown buried manholes and to incorporate them into the GIS at their proper location.
- Establish the existence of manholes in the GIS
- Identify the proper location of changes or fixtures in the system:
  - Size changes
  - Material changes
  - Angular changes
  - Tees and Wyes (sewer mains connecting without a manhole)

Inspectors also recorded changes between actual field conditions and the current GIS information on paper plots of the GIS data. The main value of this was the ability for the CCTV inspectors to validate or correct pipe connectivity throughout the Sewershed. Using these marked-up plats provided a convenient medium to record additional remarks that were then later modified in the GIS by technicians.

Smoke and dyed water testing

Smoke and dyed water testing were performed in areas where the cross-connections with storm drains were suspected and continuity of the pipe network could not be determined through other methods. Reports including photo documentation were prepared and were then used by technicians to appropriately modify the GIS data. In total, 119 smoke testing reports were generated and 24 dyed water testing reports were generated for the Outfall Sewershed.

6.3 Office Research and GIS Updates

The compilation of field collected data allowed GIS technicians to update a significant amount of the GIS representation of the wastewater infrastructure. Prioritization of the applicability of the variety of sources was performed on an attribute by attribute basis based upon the guidance provided by the City's Baltimore Sewer Evaluation Standards manual (BaSES). Some features or attributes could not be adequately quantified using the collected field information and required additional research of Baltimore's record plat maps and engineering contract drawings.

Using standard ESRI editing functionality in the ArcGIS platform as well as custom tools for GIS updates, GIS technicians utilized the sources available to them to update the wastewater geodatabase. As tiles in the City's standard grid index were completed and quality assurance approved, the data was synchronized back to the City for quality control review by the data clearinghouse.

6.4 QA/QC Review and Procedures

A variety of procedures were performed for quality assurance and quality control of the wastewater geodatabase.

- Oversight and manual spot checks by engineers and GIS analysts were performed for quality assurance.
- ArcInfo topology checks to verify feature topology; feature snapping; flow tracing; and location of duplicate features.
- Database queries to compare the GIS datasets with the other non-core data sources were executed to review for anomalies.
- An automated suite of 147 quality control tests built in the ESRI Production Line Tool Set (PLTS) platform were run against the dataset both by the sewershed consultant as well as the data clearinghouse. These tests perform a variety of checks on features and feature attributes, including: domain validation, attribute, logical, spatial, and topologic.

6.5 GIS Certification

The Outfall Sewershed team has followed the processes described above and those described in more detail in the City BaSES manual to update the City of Baltimore's wastewater GIS for the Outfall Sewershed. The City of Baltimore and the Outfall Sewershed team are hereby certifying that the GIS data represented in the Outfall sewershed portion of the City's GIS provides the necessary data for the adherence of Paragraph 14 Information Management System Program.

The Outfall Sewershed portion of the City's GIS is the best assessment of current conditions achievable with the available technology and source data. Current conditions are defined as of 01/25/2010. Furthermore, the City of Baltimore has instituted processes to ensure that should changes to the sewer infrastructure in the Outfall Sewershed occur, the GIS will be updated within 90 days of the changes.

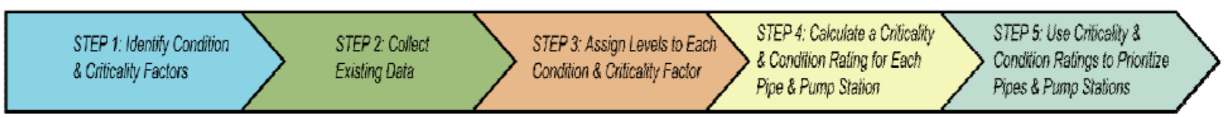
7.0 Recommendations

As required by the Consent Decree (CD), each Sewershed Study and Plan is required to identify specific improvements or other corrective actions needed to address deficiencies identified during the sewershed evaluation to aid in reducing rainfall dependent inflow and infiltration (RDI/I) contributing to sanitary sewer overflows (SSOs) or combined sewer overflows (CSOs); address deficiencies identified during the hydraulic analyses and address other deficiencies that contribute to SSOs or CSOs in the Outfall Sewershed. This section outlines how the data analysis, evaluation and the decision-making criteria were utilized to identify and prioritize improvements within the Outfall Sewershed.

7.1 Decision Making Criteria

As part of the sewershed studies, the City developed a condition and criticality protocol that provides a framework for a continuous rehabilitation strategy of all collection system components based on both criticality (consequence of failure) and condition (probability of failure). Assets whose failure can have large impacts on the community and the environment and whose condition is the poorest will receive a higher criticality rating and will receive attention sooner. Assets that receive a lower criticality rating will receive some level of continued monitoring but no immediate action or rehabilitation at this time.

The prioritization process consists of five steps illustrated below.



Step 1 - Identify the condition and criticality factors that will be used to assess the sewer system. These factors have been identified and include proximity to human population, to bodies of water, to forests, and to wildlife habitat that could potentially be affected by a sewer system failure.

Step 2 - Collect data that will be used to evaluate each factor including CCTV inspection data, manhole inspection data, pumping station inspection data, GIS data, results of hydraulic modeling, and sewer complaint data.

Step 3 - Assign different levels to each factor so that pipes, manholes, and pumping stations can be differentiated in terms of their condition or criticality.

Step 4 - Assign a condition and criticality rating for each pipe, manhole and pumping station. The ratings are assigned by using the level assigned to each factor and the relative importance of each factor.

Step 5 - Use the ratings to prioritize the system and determine short-term and long-term rehabilitation projects.

For each category, factors will be used to measure the criticality and condition of every asset. Table 7.1.1 below lists the condition and criticality categories and factors that were considered.

Table 7.1.1 – Condition and Criticality Factors

Criticality Category	Criticality Factor
Quantity of Flow Conveyed	Pipe Diameter Pumping Station Capacity
Transportation/Urban Impact	Proximity to Historic Areas Proximity to Community Areas (Parks, Schools, Etc.) Traffic Conditions Proximity to Railroad Easements
Environmental Impact	Proximity to Forested Areas Proximity to Waterways / Streams Proximity to Wetlands
Public Health Impact	Population Density Proximity to Floodplains SCADA / Warning Systems
Ease of Emergency Repair	Accessibility Ability to Re-route Flow Proximity to City Conduits Building Encroachment System Redundancy Emergency Power Ability to Bypass Flow Pipe Depth
Condition Category	**Condition Factor**
Structural Condition	Structural Pipe Rating Manhole Inspection Rating
Maintenance Frequency	O&M Pipe Rating Number of SSOs or CSOs Known Maintenance Issues Documented RDI/I Rates
Capacity	Need for Additional Capacity

Each condition and criticality factor is assigned a rating from 1 to 5. The purpose of assigning ratings to each condition and criticality factor is to differentiate sewer pipes, manholes, and pumping stations in terms of the consequences and probability of their failure.

The rating assigned increases as the consequence of failure or probability of failure increases. For example, a break in a 24-inch diameter interceptor sewer can result in more wastewater being released than a break in an 8-inch diameter collector sewer. Therefore, the larger diameter pipe has a higher criticality rating based on the amount of flow being conveyed. The 24-inch diameter interceptor sewer would be assigned a higher rating (5) for the 'Quantity of Flow Conveyed' criticality factor and the 8-inch diameter collector sewer would be assigned a lower rating (1) for the same factor.

After the individual factor ratings are assigned, an overall criticality rating and an overall condition rating is calculated for each system component. The criticality rating is calculated using the highest individual level assigned to any of the criticality factors within each Criticality Category multiplied by a relative importance value. The condition rating is equal to the highest individual NASSCO PACP or MACP rating assigned to any of the condition factors. The relative importance value for the criticality rating is the weighting, expressed as a percentage, applied to each criticality factor to calculate an overall rating. The relative importance values are the same for each collection system component and are presented in Table 7.1.2.

Table 7.1.2 – Criticality Factor Relative Importance Values

Criticality Factors	Relative Importance Value
Quantity of Flow Conveyed	30%
Transportation/Urban Impact	15%
Environmental Impact	20%
Public Health Impact	15%
Ease of Emergency Repair	20%
Total:	**100%**

The final assessment culminates in a rating of 1 through 5 for criticality and utilizing NASSCO's MACP or PACP, a 1 through 5 rating for condition, which determines priorities for repairs or continuous condition assessment or monitoring. This approach allows the City to focus their available resources and funding on the most immediate system repair needs. Figure 7.1.1 is a matrix showing the recommended course of action for each sewer system component based on the combination of condition and criticality. The vertical 1 through 5 rating scale is for condition and the horizontal 1 through 5 scale is a rating for an asset's criticality within the collection system.

Figure 7.1.1 – Condition/Criticality Matrix

<table>
<tr><th colspan="2" rowspan="2"></th><th colspan="5">Criticality</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><th rowspan="5">Condition</th><th>5</th><td colspan="5" rowspan="2">First Priority Rehab Program</td></tr>
<tr><th>4</th></tr>
<tr><th>3</th><td colspan="5">Frequent Assessment</td></tr>
<tr><th>2</th><td colspan="3" rowspan="2">Low Priority</td><td colspan="2" rowspan="2">Regular Monitoring</td></tr>
<tr><th>1</th></tr>
</table>

Second Priority Rehab Program

Each of the recommended courses of action is briefly described in more detail below. The specific improvement projects and/or other corrective actions will vary based on the type of collection system component (gravity sewer, force main, manhole, or pumping station).

First Priority Rehabilitation Program

Assets that receive a condition rating of 5 regardless of criticality, and assets that receive a condition rating of 4 and criticality rating of 4 and 5 are placed at the highest priority for rehabilitation, repair or replacement. These assets lack hydraulic capacity, contribute to system inflow and infiltration (I/I) and/or are likely to fail in the near future. They present the potential for SSOs or could create a major disruption in service and potentially impact the environment and/or public health if not addressed.

Second Priority Rehabilitation Program

Assets that receive a condition rating of 4 and criticality rating of 1, 2, or 3 will be given second priority in the rehabilitation program. These assets contribute to system I/I, and are likely to continue to deteriorate and to require attention in the foreseeable future.

Frequent Assessment

Assets that are in fair physical condition (PACP/MACP condition rating of 3), should have their condition assessed frequently, every 2 to 3 years regardless of the criticality rating. The purpose of frequent assessment is to check if the condition has deteriorated to a point that the asset would need to be moved to a higher priority.

Regular Monitoring

The assets in the regular monitoring category are typically in serviceable condition (PACP/MACP condition rating of 1 or 2), but received a high criticality rating of 4 or 5. These assets should be checked every 3 to 5 years.

Low Priority

The low priority category includes assets that are believed to be in good condition (PACP/MACP condition rating of 1 to 2), and received a lower criticality rating of 1 through 3. The assets in this category will receive some level of inspection (once every 5 to 10 years) to verify that their conditions are not continuing to deteriorate.

7.2 Proposed Improvements

The proposed improvements address structural deficiencies and hydraulic restrictions in the Outfall Sewershed.

Once the sewer system improvement projects and/or other corrective actions required to address deficiencies were identified and ranked based on the criticality and condition ratings; assets that received a condition rating of 5, regardless of criticality, were included in a “First Priority” corrective action plan. Assets that had a condition rating of 4 and a criticality rating of 4 or 5 were also included in a “First Priority” corrective action plan. Assets that received a condition rating of 4, but were not considered to be as critical (3 or less) were included in the “Second Priority” corrective action plan.

Asset prioritization was developed with consideration that all proposed improvement projects required to eliminate SSOs must be completed before January 01, 2016 as stipulated by the CD. These assets included First and Second Priority manholes and sanitary sewers, identified SSO structures, and recommended hydraulic improvements to the collection system. These proposed improvement projects are as follows:

7.2.1 Sanitary Sewer Overflow Structure Identification and Elimination

As a requirement of the City’s CD, the Sewershed Study and Plan is required to identify undocumented SSO structures. For the 2-year return period event, there are four undocumented SSO locations in the simulation results with Future 2025 conditions and the upstream improvements in place. The largest volume SSO is at Durham and Eager Streets near the downstream end of the 99-inch sewer. Two SSO locations are at the upstream end of the 99-inch sewer along Bond Street between Fayette and Orleans Streets. Another SSO location, at Bethel and Moyer Streets, is on a small branch sewer that connects to the upstream end of the 99-inch sewer. These overflow locations are discussed in more detail in Section 5.3.3 and in the BACA report, Attachment 5.3.1.

7.2.2 Structural Deficiencies Identified

Proposed Manhole Improvements

Table 7.2.1 shows a listing of all manholes inspected within the Outfall Sewershed based on the results of the Condition and Criticality rankings. The manholes that received a final assessment of “First Priority” or “Second Priority” are recommended for repairs as part of the rehabilitation program.

Table 7.2.1 – Manholes Condition and Criticality Assessment (For Manholes Located on Small and Large Diameter Sewers)

Final Condition and Criticality Assessment	Small Dia.		Large Dia.		Total MHs	
	Count	%	Count	%	Count	%
Low Priority	111	5.7%	11	19.0%	122	6.0%
Regular Monitoring	0	0.0%	0	0.0%	0	0.0%
Frequent Assessment	1,529	77.9%	21	36.2%	1,550	76.7%
Second Priority Rehabilitation	16	0.8%	0	0.0%	16	0.8%
First Priority Rehabilitation	5	0.3%	0	0.0%	5	0.2%
“Could Not Locate” (CNL) Manholes	302	15.4%	26	44.8 %	328	16.2%
TOTAL MANHOLES	**1,963**	**100%**	**58**	**100%**	**2,021**	**100%**

CNL Manholes were not assigned a Condition and Criticality Assessment Score as no valid condition data was available. Manholes that were identified as abandoned or non-sewer assets are not included in the totals.

Proposed Sanitary Sewer Improvements

Table 7.2.2 shows the length of the sanitary sewers located within the Outfall Sewershed and their respective Condition and Criticality rankings. The PACP quick ratings for each pipe were converted to a single pipe rating by summing the segment grade scores for the two highest defect grades per the methodology used in the BaSES manual.

Each sewer receiving an initial First or Second Priority ranking was then examined in detail to determine the proper rehabilitation technique for each pipe segment. This review identified numerous sewers that were identified as First or Second Priority sewers that had been placed in those categories due to types of defects that could not be directly addressed using point repairs or cured-in-place-pipe lining, such as settled deposits, minor obstructions and other defects that would be addressed through a system cleaning. Many lengths were identified in this evaluation and were re-rated as Frequent Assessment lengths and should be inspected according to the requirements of that category. Similarly, sewers that were initially rated Low Priority, Regular Monitoring or Frequent Assessment were screened to determine if any of these sewers had specific defects that could escalate to a pipe failure condition. Several lengths were identified with these serious defects and were re-rated as First Priority lengths.

Table 7.2.2 – Small and Large Diameter Sanitary Sewers - Condition and Criticality Assessment						
Final Assessment	**Small Dia.**		**Large Dia.**		**Total Sewers**	
	LF	**%**	**LF**	**%**	**LF**	**%**
Low Priority	78,065	25.7%	9,576	37.9%	87,641	26.6%
Regular Monitoring	25,033	8.2%	15,594	61.7%	40,627	12.3%
Frequent Assessment	104,434	34.4%	0	0%	104,434	31.7%
Second Priority Rehabilitation	90,851	29.9%	97	0.4%	90,948	27.6%
First Priority Rehabilitation	5,603	1.8%	0	0%	5,603	1.7%
Total LF	**303,985**	**100%**	**25,267**	**100%**	**329,252**	**100%**

Note that the total lengths included here do not include pipes which will not be inspected due to easement considerations, were abandoned, were identified as non-sewer assets or other approved reasons.

Outfall Interceptor, Outfall Relief Sewer and 99-inch Sewer Pipelines

Based on our review of the CCTV Inspection tapes, the Joint Venture did not detect any Grade 5 structural deficiencies in these three large diameter pipelines. However, as previously discussed, areas of exposed and missing aggregate were noted along the majority of reaches of these three pipes. The majority of the large diameter pipes are unreinforced concrete, thus it was not possible to accurately determine from the CCTV tapes the extent of loss of wall material.

One reach along Durham Street (Sewer ID S45CC_058G1) has a cross-section comprised of a brick “U: shape for lower half, and a concrete top slab. In the spring of 2009 continuing pavement deterioration was noted near the intersection of Durham and Eager Streets. As a result, this segment was re-inspected by CCTV. Areas of exposed rebar in the top slab were noted in the CCTV video. The Joint Venture, evaluated this sewer segment, and discussed recommendations with the City in December 2, 2009. This segment scored a Grade 4 condition rating (due exposed rebar, exposed and missing aggregate), and a Grade 4 criticality rating. The combination identifies it as a project to include in the First Priority Rehabilitation Program.

Overall, these pipes were rated a grade 3 for the structural evaluation. However, for the maintenance portion of the assessment three segments were given a Grade 5, approximately half the segments were given a Grade 4, and the remainder a Grade 3 due to the amount of sediment / debris buildup in the sewers. As discussed in Section 5, the significant sediment deposition has decreased the hydraulic capacity of all three large diameter lines.

Two courses of action are recommended for these three large sewers:

1. The lack of hydraulic capacity in the Outfall Interceptor, Outfall Relief Sewer and 99-inch Sewer Pipelines are recommended to be cleaned as a “First Priority Rehabilitation Program within the next 2 to 3 years. However, it should be coordinated with any proposed downstream improvements at the WWTP so that re-sedimentation does not occur before hydraulic restrictions

are remedied. This task is listed hereinafter as a hydraulic Improvement, and the costs are included in Table 7.4.3.

2. Repair the top slab and reline Sewer ID S45CC_058G1. This is identified for inclusion in the First Priority Rehabilitation Program, The cost for rehabilitation for this segment is included in Table 7.4.2

7.2.3 Proposed Outfall Sewershed Collection System Hydraulic Improvements

The interrelationships between the Outfall Sewershed and the other upstream and downstream sewersheds have a significant influence on the characteristics of the hydraulic improvements to be discussed below for the Outfall Sewershed. The Jones Falls, Herring Run, High Level, Low Level, and Dundalk sewersheds flow into the Outfall sewershed. The flow from Baltimore County enters the collection system downstream of the Outfall Sewershed. These sewersheds are hydraulically interdependent. Boundary conditions have been defined for independent hydraulic modeling of each sewershed. The technical program manager is in the process of evaluating the collection system with all of the sewersheds integrated into the single system-wide Macro hydraulic model. The Outfall Sewershed Plan contains certain recommended improvements that would be implemented by the City in accordance with a proposed schedule. However, the Plan should not be considered final and may require amendment based on results of the system-wide Macro hydraulic model.

For the 2-year storm, 29 million gallons of SSOs are predicted for the Outfall Sewershed.

The 2-year, 24-hour storm event is used to establish the hydraulic improvements that are required to eliminate SSOs in the Outfall Sewershed. Details on improvements required to address SSOs for other return period storm events can be found in Section 5.0 and in Attachment 5.4.1 – Outfall Sewershed Alternatives Analysis and Recommendations Report (AARR).

As discussed in Section 5.0, the major hydraulic improvement to be implemented in the Outfall Sewershed for the 2-year return period event is heavy sediment cleaning in the large diameter trunk sewers. Downstream of the Outfall Sewershed, other improvements for the 2-year event are also required to achieve the 2-year level of protection in the Outfall Sewershed.

In the Outfall Sewershed there are 28,000 LF of large diameter trunk sewers in the City of Baltimore. The inspection revealed that the sediment depth averages approximately 30-inches in the Outfall Interceptor. Similar sediment depths are present in the 99-inch sewer and the Outfall Relief sewer. An estimated 34,000 tons of sediment are to be removed. Being proactive, the City has already begun an evaluation to determine the highest priority areas along the Outfall Sewer for sediment removal and intends to implement a design contract and one or more construction contracts to expedite the sediment removal process

The 2-year level of protection is contingent on the following assumptions:

- The inflow boundary conditions into the Outfall Sewershed are an accurate prediction of the flows from the upstream sewersheds once the hydraulic improvements are implemented in the upstream sewersheds. The upstream improvements involve conveyance restoration by sediment removal, conveyance capacity enhancements by sewer replacement or relay, peak flow attenuation using storage tanks, and infiltration and inflow reduction by sewer and manhole rehabilitation;
- The Eastern Avenue Pump Station does not discharge more than 108 MGD (3 pumps online). This means that the pump station will not operate with all pumps on line for the 2-year event;
- After cleaning sediment, the Manning's roughness coefficient is 0.015 or less for the large diameter trunk sewers; and
- Downstream improvements increase the conveyance capacity to the Back River WWTP. The downstream improvements include cleaning sediment from the Outfall Interceptor and Outfall Relief sewer in Baltimore County and upgrades to the WWTP (for details see Section 7.2.4 below). This means that the maximum water level in the pipes at the County Line does not exceed 48 feet above datum (so that the pipes are not surcharged and there is no less than 1 ft of head space from the maximum water levels to the crowns of the pipes).

Along with the major hydraulic improvements listed above, a short-term improvement project is recommended to reduce the risk of overflows at Bethel Street and Moyer Street (manhole S43E__016MH), between Baltimore Street and Fairmount Street. This project is intended to attenuate the problem while waiting for the major hydraulic improvement to be implemented.

Peak wet weather discharge rates from the Eastern Avenue Pump Station exceed the conveyance capacity of the 99-inch sewer. The SSO at Bethel Street is a result of surcharging that occurs in the 99-inch sewer due to the rate of sewage being discharged from the Eastern Avenue Pump Station. Surcharging in the 99-inch sewer reverses the flow in the 24-inch branch sewer and is relieved by overflowing at the Bethel Street manhole, which has the lowest rim elevation in the vicinity.

To address the problem, the City is contemplating the hydraulic separation of the 24-inch branch sewer from the 99-inch sewer. This would be accomplished by diverting the flows in the 24-inch branch sewer to the adjacent Low Level Sewershed. The City has begun supplemental flow monitoring in order to characterize the peak wet weather flows in the area tributary to the 24-inch branch sewer, and will use the hydraulic model to evaluate different options to accomplish this. The additional flow monitoring and model simulations will allow the City to assess what impact, if any, this short-term measure will have on the Low Level system, and to identify additional RDII reduction projects in the tributary area as necessary.

7.2.4 Upgrades to Headworks at the Back River WWTP (BRWWTP)

The development of the Macro model revealed a hydraulic restriction created by the headworks at the BRWWTP. After the flows pass through the headworks screens, the influent channel to the grit tanks has an adverse slope, rising vertically nearly 4 feet. To exacerbate the issue, the Outfall Interceptor system is on a relatively flat slope throughout the City and County to the treatment plant. The invert elevation of the Outfall Interceptor system at the City-County boundary was compared to the grit tank effluent weirs (which are set at the lowest possible elevation). The crest elevation of the effluent weirs is 4.31 feet higher than the invert elevation of the Outfall Interceptor at the City-County boundary, which is a distance of approximately 10,000 feet upstream of the BRWWTP. This hydraulic control creates a backwater conditions for miles upstream, causes the accumulation of debris in the Outfall Interceptor and Outfall Relief Sewer.

The final selection of the improvements necessary to alleviate the hydraulic restriction at the BRWWTP is beyond the scope of this study. This is a complex process that requires further in-depth analysis by the City and discussion with MDE and EPA. Approaches to address the capacity limitation may include the construction of flow equalization facilities or expanded treatment capacity at the BRWWTP. Nonetheless, this study recognizes the need to eliminate the hydraulic restriction at the BRWWTP in order to eliminate SSOs, and account for these improvements in its recommendations.

The City is currently proceeding with the planning and preliminary design of wet weather management facilities and removal of the hydraulic constraint at the BRWWTP. However, the design of any improvements at the BRWWTP cannot be finalized until the Baltimore County Wet Weather Consent Decree planning is completed and accepted by EPA and MDE. Upon completion and approval of the County's Wet Weather Plans, it is anticipated that the City can finalize the design of the improvements at BRWWTP, obtain MDE's approval and implement these projects within a 6 to 8 year period.

7.3 Proposed Improvement Implementation Schedule

An implementation schedule for completion of the proposed SSO elimination and sewer system improvements has been developed as part of this project based on project cost, anticipated project duration, available manpower, and materials. In all cases, projects have been scheduled to minimize public impact and coordinated with other similar projects being conducted throughout the City. The implementation schedule was developed with consideration that all proposed improvements must be completed before January 1, 2016 as stipulated by the CD, with the exception of the improvements at the BRWWTTP as indicated above.. The following schedules have been developed providing time to successfully complete the required work.

Manhole Rehabilitation:

The schedule provided in Table 7.3.1 represents a reasonable duration required for the City to select an engineering consultant to prepare the required design documents, advertise the project, select a contractor to complete the required repairs and have the effectiveness of the repairs evaluated.

Note that all First Priority and Second Priority Manholes are located on small diameter sewers. No manholes requiring immediate attention were found on the large diameter trunk sewers.

Table 7.3.1 – Manhole Rehabilitation Implementation Schedule

Paragraph 9 Project	Project	Description	CD Milestone Dates		
			Advertise Project	Construction Complete	Evaluation Phase Completion
1	Sanitary Sewer Manholes - First Priority Rehabilitation	Completion of First Priority Manhole Rehabilitation/ Replacement Projects	1/1/2012	12/31/2013	12/31/2014
2	Sanitary Sewer Manholes - Second Priority Rehabilitation	Completion of Second Priority Manhole Rehabilitation/ Replacement Projects	1/1/2013	12/31/2014	12/31/2015

Sanitary Sewer Rehabilitation:

The schedule provided in Table 7.3.2 represents a reasonable duration required for the City to select an engineering consultant to complete the required design documents, advertise the project, select a contractor to complete the work and have the effectiveness evaluated.

Table 7.3.2 – Sanitary Sewer Rehabilitation Implementation Schedule

Paragraph 9 Project	Project	Description	CD Milestone Dates		
			Advertise Project(s)	Construction Complete	Evaluation Phase Completion
3	First Priority Rehabilitation	CIPP, Point Repairs, and Combination CIPP/Point Repairs for First Priority	1/1/2012	12/31/2013	12/31/2014
4	Second Priority Rehabilitation	CIPP, Point Repairs, and Combination CIPP/Point Repairs for Second Priority	1/1/2012	12/13/2014	12/31/2015

Hydraulic Improvements:

The schedule provided in Table 7.3.3 represents a reasonable duration for the City to select an engineering consultant, complete the required design documents, advertise the project, select a contractor, implement the required improvements, and evaluate the effectiveness of the repairs. The improvements are the sediment removal recommended to prevent SSOs in the 2-year storm event.

Table 7.3.3 – Hydraulic Improvement Schedule

Paragraph 9 Project	Project	Description	CD Milestone Dates		
			Advertise Project	Construction Complete	Evaluation Phase Completion
5	Bethel Street SSO Reduction	Diversion to Low Level Sewershed	1/1/2012	6/30/2013	6/30/2014
6	Sediment Cleaning Trunk Sewers in City of Baltimore	Removal of Sediment and Debris from the 99” sewer, Outfall Interceptor and Outfall Relief sewer	12/31/2011	12/31/2014	12/31/2015
7	Sediment Cleaning Trunk Sewers in Baltimore County	Removal of Sediment and Debris from the Outfall Interceptor and Outfall Relief sewer	6/30/2013	12/31/2014	12/31/2015

7.4 Estimated Costs of Proposed Improvement Projects

To characterize expected costs for the collection system improvements necessary in the Outfall Sewershed, the City completed a review of information compiled from prior City projects for various types of repairs, rehabilitation and replacement of manholes and sanitary sewers. In addition costs targeted specifically for large diameter sewer construction, were developed as part of the Outfall Sewershed AARR. Once compiled, the information was reviewed, compared and normalized for use in preparing reasonable estimates for the City’s sewershed improvements.

The prices utilized in estimating these costs represent average unit costs that were derived from the sources identified. There was, however, some significant variability noted when comparing the unit costs developed by contractors bidding on the same project, and there was considerable variability when comparing these documented unit costs with other similar types of repair techniques employed on different projects. Such unit cost variability reflects both the site specific nature of each project as well as the normal variability typically associated with varying markets, project time constraints and other construction related considerations. While it is understood that site specific attributes will have an impact on final costs for a given rehabilitation/repair/replacement effort, it is the City's intent to ensure that all of the sewersheds use the same baseline cost assumptions for consistency and planning purposes. These fully-loaded costs are an attempt to capture all the relevant costs associated with a construction project such as mobilization, bypass pumping, site/paving restoration, and repair of other utilities, which can add significantly to the cost, but are typically required to complete the overall project.

7.4.1 Estimated Improvement Budget

The following section outlines the proposed costs utilizing fully-loaded cost data required to implement the First and Second Priority collection system improvements.

Estimated Manhole Rehabilitation Budget

Table 7.4.1 summarizes the estimated 2008 costs required to rehabilitate all First and Second Priority sanitary sewer manholes identified in the Outfall collection system.

Table 7.4.1 – Estimated Manhole Rehabilitation Improvement Budget				
First Priority Manholes				
Item	**Method**	**Unit Cost**	**Quantity (ea.)**	**Cost**
Manhole	Rehabilitation/Replacement	$3,719	5	$18,595
Design, Const. Mgmt./Insp. Etc. (42%):				$7,810
Total - First Priority MH's				$26,405
Second Priority Manholes				
Manhole	Rehabilitation/Replacement	$3,719	16	$59,504
Estimated Design, Const. Mgmt./Insp. Etc. (42%):				$24,991
Total - Second Priority MH's				$84,495
Total Estimated First and Second Priority Manholes:				**$110,900**

Estimated Sanitary Sewer Rehabilitation Budget:

Table 7.4.2 summarizes the estimated 2008 costs required to rehabilitate all First and Second Priority sanitary sewers identified in the Outfall Sewershed collection system.

Table 7.4.2 – Estimated Sewer Rehabilitation and Replacement Improvement Budget

First Priority Sewers					
Sewer Size	**Unit Cost (s)**		**Quantity (LF)**		**Cost**
CIPP Lining					
8” Sewer Lining	$45		275		$12,375
8+" - 12" Sewer Lining:	$64		343		$21,952
12+" - 18" Sewer Lining:	$87		1,799		$156,513
18+” – 24” Sewer Lining:	$124		950		$117,800
Total Small Diameter CIPP Lining:			3,367		$308,640
99-inch Sewer Lining	$750		0		$0
Total Large Diameter Lining			0		$0
Total Small and Large Diameter Lining			3,367		$308,640
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$129,629
Total First Priority Small Diameter CIPP Lining & Large Diameter Lining:					**$438,269**
Sewer Replacement					
8” Sewer Replacement:	$150		0		$0
8+" - 12" Sewer Replacement:	$275		216		$59,400
12+" - 18" Sewer Replacement:	$325		293		$95,225
18+” – 24” Sewer Replacement:	$600		358		$214,800
> 24” Sewer Replacement:	$ 800		373		$298,400
Total Replacement:			1,240		$667,825
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$280,487
Total First Priority Small Diameter Sewer Replacement:					**$948,312**
Point Repair & CIPP Lining					
	Point Repair	CIPP	Point Repair	CIPP	
8" Point Repairs/CIPP:	$378	$87	25	591	$60.867
8+” - 12" Point Repairs/CIPP:	$378	$87	20	358	$38,706
Total Point/CIPP Repairs:			45	949	$99,573
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$41,821
Total First Priority Small Diameter Point/CIPP Repairs:					**$141,394**
Total Estimated First Priority Sewers:					**$1,527,975**

Table 7.4.2 – Estimated Sewer Rehabilitation and Replacement Improvement Budget (continued)

Second Priority Sewers					
Sewer Size	**Unit Cost (s)**		**Quantity (LF)**		**Cost**
CIPP Lining					
8" Sewer Lining:	$45		77,208		$3,474,360
8+" - 12" Sewer Lining:	$64		1,709		$109,376
12+" - 18" Sewer Lining:	$87		2,018		$175,566
18+" – 24" Sewer Lining:	$124		0		$0
> 24" Sewer Lining:	$160		1,032		$165,120
Total CIPP Lining:			81,967		$3,924,422
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$1,648,257
Total Second Priority Small Diameter CIPP Lining:					**$5,572,679**
Sewer Replacement					
8" Sewer Replacement	$150		216		$32,400
Total Sewer Replacement:			216		$32,400
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$13,600
Total Second Priority Small Diameter Sewer Replacement:					**$46,000**
Point Repair & CIPP Lining					
	Point Repair	CIPP	Point Repair	CIPP	
8" Point Repair/CIPP:	$378	$45	530	8,292	$573,480
8+" – 12" Point Repairs/CIPP:	$378	$64	10	196	$16,324
12+" – 18" Point Repairs/CIPP:	$378	$87	10	324	$31,968
Total Point/CIPP Repairs:			550	8,812	$621,772
Estimated Design, Const. Mgmt./Insp. Etc. (42%):					$261,144
Total Second Priority Small Diameter Point/CIPP Repairs:					**$882,916**
Total Estimated Second Priority Sewers:					**$6,501,595**
Total Estimated First and Second Priority Sewers:					**$8,029,570**

Estimated Hydraulic Improvement Budget:

Table 7.4.3 contains the estimated 2008 costs required to complete the hydraulic improvements for the 2-year storm event in the Outfall Sewershed. Also included in the table is an estimate of the tonnage of sediment and the cost to remove sediment in Baltimore County.

Table 7.4.3 – Estimated Hydraulic Improvement Budget							
2-year Outfall Improvements							
Site	Improvement	Unit Cost		Quantity		Cost
Sediment Cleaning in Trunk Sewers **Outfall Sewershed (within the City of Baltimore)**						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29,000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$17,100,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$7,200,000
2008 Total Estimated Cost in the City						**$24,300,000**
Sediment Cleaning in Trunk Sewers **Downstream of the Outfall Sewershed (in Baltimore County)**						
Sediment Removal in the Outfall Interceptor and Outfall Relief Sewer in the County (Remove 29,000 tons from 21,000 LF of pipe) Cost includes 42% City mark-up for engineering. design, construction management/inspection, administration, post-engineering services, contingency						**$20,600,000**
Total Cost of Sediment Removal in the City and the County						**$44,900,000**
Bethel Street SSO Reduction Improvement						
Site	Improvement					Cost
24-inch branch sewer	Diversion to Low Level					**$350,000**

7.5 Sewershed Re-Inspection Program

Per the requirements of the CD, the City's Outfall Sewershed collection system needs to be re-inspected by January 1, 2016. The following sections outline the requirements of the re-inspection program and provide a general schedule to complete this work.

7.5.1 Re-Inspection Prioritization Scheme

The City's condition and criticality protocol provides a framework for a continuous rehabilitation strategy of all collection system components based on both criticality (consequence of failure) and condition (probability of failure). Assets whose failure can have large impacts on the community and the environment and whose condition is the poorest will receive a higher criticality and condition rating and will receive attention in a more timely manner. Assets that receive a lower criticality and condition rating will receive some level of continued monitoring as recommended herein but no immediate action or rehabilitation. Refer to Section 7.1 Decision Making Criteria for details. The following sections detail the requirements of future inspection programs.

7.5.2 CCTV and Manhole Inspections

The implementation schedule provided includes provisions for the re-inspection of each of the Outfall Sewershed collection system components by January 1, 2016. The proposed re-inspection schedule includes provisions for, but is not necessarily limited to, a prioritization scheme for further inspection of collection system components based on the following criteria:

1) Prior identification of system defects, prior NASSCO PACP or MACP rating codes, grease blockages, root intrusion or system complaint data.
2) Prior criticality and condition ratings.
3) Expected life cycle of system components.
4) Estimated rate of existing or potential inflow and/or infiltration.
5) Scheduled rehabilitation or other corrective action of a system component; and the predetermined re-inspection frequency of a collection system component.

Current sewershed studies are scheduled to be completed by July 2010. Following these studies, the City intends to implement a continuous CCTV and Manhole Inspection program aimed at re-inspecting all gravity sewers 8-inches and larger, associated manholes and other sewer structures by January 1, 2016. The planned re-inspection activities will be prioritized based on the condition and criticality factors determined during this project.

Based on the results of the inspections completed during this sewershed study, the re-inspection schedule identified by the CD and the rehabilitation work which is described as part of this plan, it is recommended that all PACP condition grade 1 and 2 sewers in the Outfall Sewershed be re-inspected in a 5 to 10 year range. All PACP condition grade 3 sewers should be re-inspected in 2 to 3 years to reassess their condition and assign appropriate repairs as needed.

The implementation schedule for re-inspection of these sewershed system components is outlined in Table 7.5.1.

Table 7.5.1 – Outfall Sewershed Re-Inspection Implementation Schedule

Task	Duration (Yrs.)	Start Date	End Date	2011	2012	2013	2014	2015	2016
Manhole Inspections	3 1/2	1/1/2011	6/30/2014						January 1, 2016
Analysis and Report	1 1/2	7/1/2014	12/31/2015						
Sewer Inspections	3 1/2	7/1/2012	12/31/2014						
Analysis and Report	1 1/2	7/1/2014	12/31/2015						

Based on the condition of the assets observed during this study, manholes and sewers that received higher condition and criticality rating scores were recommended for inclusion on the First and Second Priority corrective action plan. Once rehabilitated, these manholes and sewers should be placed on a "Low Priority" inspection program with regular inspections occurring once every 5 to 10 years.

The manholes and sewers that received condition ratings of 3 were classified as requiring "Frequent Assessment" under the condition and criticality rating system and should be inspected on regular 2-3 year inspection intervals to ensure the continuity of the collection system.

Manholes and sewer segments that received a rating of 2, (identified as requiring "Regular Monitoring") should be inspected every 3-5 years. Based on the results of those inspections, any manholes and/or sewers that have continued to deteriorate to a point that requires repair should be repaired on an as-needed basis to address specific problems or deficiencies that have occurred.

As part of the ongoing manhole inspection program, it is recommended that a field crew be assigned to investigate manholes that could not be located during this study and to inspect them if possible. These manhole inspections are included in the hydraulic upgrade costs and are identified in the hydraulic improvements schedule. To date, 213 manholes were designated as "CNL" (could not locate) and are shown on Table 7.2.1 and listed in Appendix 4.2.1.

7.6 Future Data Collection and Evaluation Services

As required by the CD, under Paragraph 9-C-xii, the City will be required to implement several continuous data collection programs in order to assess the effectiveness of the rehabilitation programs and other O&M enhancement efforts within the sewershed. These programs will be comprehensive, system-wide initiatives that will include a long-term flow monitoring plan, a sewer cleaning program, CCTV and manhole inspection programs and root control and grease control programs. These are discussed in more detail in the following sections.

7.6.1 Long-Term Flow Monitoring Plan

In 2006 the City of Baltimore implemented a comprehensive flow monitoring program for the purpose of evaluating the severity of infiltration and inflow and for calibration of the hydraulic model. This comprehensive program consisted of a network of about 350 flow meters, 20 rain gauges, and 33 groundwater monitoring stations and extended for a period of one year from May 2006 through May 2007. In May 2007, the network was reduced to about 100 flow meters that were placed at key points and junctions in the collection system for the purpose of long term assessment and continuous calibration of the hydraulic model. All 20 rain gauges remained in operation. The City plans to continue monitoring the flows in order to assess the effectiveness of the on-going and future rehabilitation and O&M enhancement programs.

7.6.2 Small Diameter Sewer Cleaning Program

As part of the sewer inspection program completed for this study, small diameter branch sewers that were inspected were also cleaned. The sewers were cleaned so the inspections could identify defects that would not otherwise be visible during the inspection and to remove debris from the sewer to restore at least 95% of the original conveyance capacity. When significant restrictions such as roots or other debris were encountered, heavy cleaning was utilized to restore the capacity of the sewers and allow for internal inspection. Heavy cleaning involved root cutting, grease removal and/or additional passes of the hydro-cleaning equipment to remove heavy accumulations of sediment and debris. All debris was removed from the sewers and disposed of at an approved disposal site. When significant blockages were encountered that could not be addressed by cleaning operations, they were reported to the City and the City promptly coordinated with the wastewater maintenance division or their on-call contractor to resolve the deficiency.

Based on the cleaning work completed during this project and observations from the inspection work completed, it is recommended that sewers which contain heavy accumulations of grease, debris and/or roots, sewer siphons, and sewers with velocities less than 3 feet per second (fps) should be cleaned on a 5-year interval. These cleaning operations should be closely coordinated with the sewer re-inspection program, which needs to be completed by January 1, 2016 and prioritized based on condition and criticality rating factors that were determined during the inspections described in Section 7.1. Under normal operating conditions, the remaining sewers should follow the 5 to 10-year Low Priority sewer inspection cycle.

7.6.3 Large Diameter Sewer Cleaning Program

The large diameter trunk sewers were not cleaned in the sewer inspection program. Heavy cleaning is recommended in the hydraulic improvements section above. In the future, the need for ongoing cleaning and the frequency of cleaning the large diameter trunk sewers is unknown. The risk of re-accumulating sediment will depend on hydraulic conditions after downstream improvements at the WWTP are in place.

Downstream improvements, such as an influent pump, can lower the water level at the headworks to the plant. Once the specific facilities of downstream improvements are defined, the impact on velocities in the trunk sewers can be determined for dry and wet weather conditions.

A specific study of the sediment transport process is recommended to estimate the need for and frequency of on-going sediment cleaning. The study of sediment transport should involve the following components:

- a characterization of the sediment type (range of particle sizes, densities, and cohesiveness),
- an estimate of sediment loading rates from the upstream sewersheds (dry and wet weather rates of delivery to the Outfall Interceptor),
- a hydraulic/sediment transport simulation model

With the model, the sediment transport in the trunk sewers can be simulated to estimate the delivery of sediment to the WWTP. The hydraulic/sediment transport simulations should account for time variable deposition, re-suspension, and transport dynamics in response to variable flow rates in dry and wet weather. A recommendation about the need for and frequency of future sediment cleaning efforts can be based on the results of this study.

7.6.4 CCTV and Manhole Inspection Programs

The City also intends to implement continuous citywide CCTV and manhole inspection programs following the completion of the sewershed studies, which are scheduled to be completed by July 2010. These programs will be aimed at re-inspecting all gravity sanitary sewers 8-inches and larger in diameter, force mains, pumping stations, manholes and other sewer structures by January 1, 2016. There are no pumping stations and force mains in the Outfall Sewershed, so this program will focus on the sewers and manholes identified in the evaluation process. The planned re-inspection activities will be prioritized based on each segment's condition and criticality ratings that were derived during the sewershed inspections described in Section 7.1 of this report.

7.6.5 Root Control Program

In 2004, under Project 1015, the City began monitoring the impacts of root infestation in their collection system by tracking and geocoding customer calls related to root problems in the sewer. In 2006, the City identified an area in the Herring Run Sewershed having severe root intrusion problems (approximately 1,500 acres, 230,000 linear feet of pipe). The City proceeded to implement a root control chemical application pilot project in this area in 2007, which included the treatment of approximately 150 house laterals and service connections. The pilot project yielded promising results. The City is therefore expanding the Root Control Program (RCP) into other areas of the collection system with documented root intrusion problems. A recent evaluation of customer calls in 2007 identified two additional areas with severe root infestation (see Figure 7.6.1).

To evaluate the effectiveness of the on-going root control program, the Project 1015 Technical Manager will use other sources of information, such as CCTV and manhole inspections, will be used to validate and direct the root control efforts. The goal of the on-going RCP is to treat all areas of the collection system experiencing root infestation once every three to five years. The effectiveness of the RCP will be assessed by continued monitoring of the areas and continuous evaluation of customer complaint calls within these areas on a six month review basis.

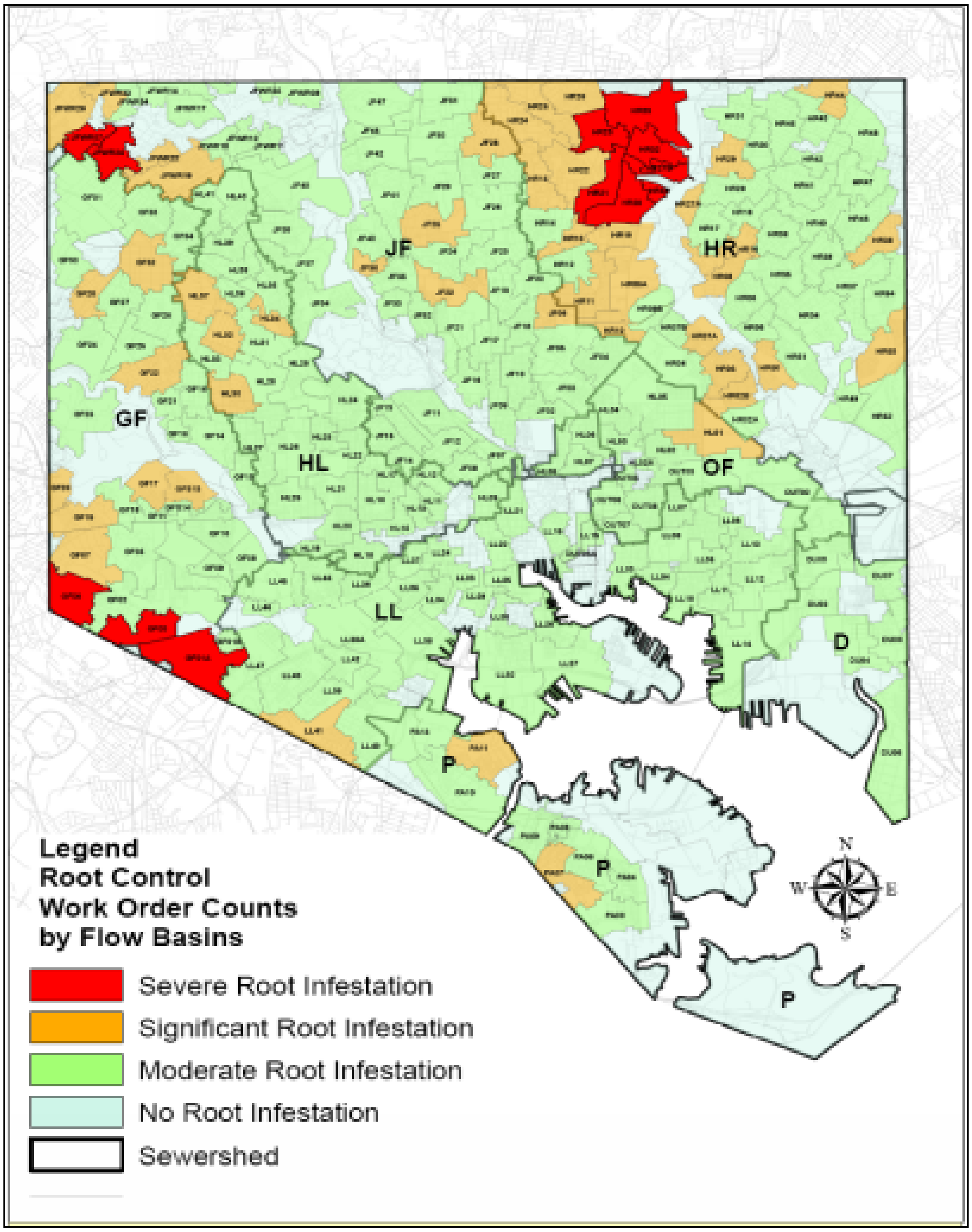


Figure 7.6.1 – Root Control Analysis

7.6.6 Fats, Oils and Grease Control Programs

Similar to root infestation in the sewer system, the City, under Project 1015, began assessing the impacts of Fats, Oils and Grease (FOG) in the collection system in 2004. The City geocoded and mapped all customer complaint calls related to FOG and identified five sections of the collection system where severe problems exist. Not surprisingly, these sections serve areas with numerous restaurants and/or food establishments, namely Little Italy and the Johns Hopkins Hospital area - where many restaurants serve the hospital community, and the upper reaches of the High Level Sewershed, which have numerous restaurants and a major mall with a food court. The City proceeded to outfit two of its newest sewer vac-trucks with de-greasing equipment and began treating the targeted areas in 2006. These areas are currently on a regular cleaning schedule and are addressed twice a year for grease. Baltimore will continue to evaluate customer complaint calls and utilize CCTV and manhole inspection data in order to assess and guide future activities of the FOG Program.